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CCV.

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THE LOCATION OF THE CHIMBOTE TUNNELS.

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The Chimbote and Huaraz Railway was projected for the relief of the upper valley of the Santa River; a valley proverbial for the beauty of its scenery, for its mineral and agricultural wealth, and for the healthfulness of its climate.

Raised from 6 000 to 12 000 feet above the sea, this valley is relieved from the intense heat of the coast, while from its eastern boundary, the snow range of the Cordillera, comes an abundant supply of water for the irrigation ever necessary on the western slope of the Peruvian Andes.

In this valley, within a distance of 60 miles, six large towns have grown up with populations which may be placed as follows :

Huaylas.....	5 000	Caraz.....	6 500
Yunguay.....	4 000	Carhuaz.....10 000	
Huaraz.....15 000		Recuay	4 000

The entire population of the valley cannot be less than 70 000 souls. These people have established close intercourse with their neighbors in the more extended valley of the Marañon, but their only intercourse with the capital of the country and with the smaller towns on the coast, is conducted by mule traffic over the high passes of the coast range, for the Santa leaves the valley through a succession of gorges beginning with the Cañon of Pato and ending at Infernillos, thirty miles distant. These gorges rested in undisturbed tranquility till 1873, and to-day it may be said that no living mortal has been through the Cañon of Pato.

A preliminary study for a railway was authorized by the Government in 1864, but was not completed until some years later, when a line was projected beginning at Chimbote Bay, where a harbor of greater safety, if not of easier entrance, could be found than at the old port of Santa.

Traversing, first, the lands of the Chimbote valley—deserted two or three centuries ago for want of water for irrigation—the line enters the valley of the Santa river, which it follows, first, through tillable land, with here and there a sugar estate, then through the gorges before referred to, ending at Recuay, 265 kilometers (164 miles) from Chimbote. The survey or study was made for a broad gauge road, and the estimate of cost was 38 000 000 soles. For the purposes of this paper I shall consider soles the equivalent of dollars, since this was about their relation at the period of which I write.

The estimated cost was thought too great, and a narrow gauge road was decided upon. The reduction in the estimated cost seems to have been proportioned, judging from the figures, to the reduction in gauge, a method of great simplicity, at least.

The contract for location and construction was let in November, 1871, to Don Enrique Meiggs for about 24 000 000 soles. The specifications were quite liberal, providing for curves of minimum radius of 61 meters (200 feet), and permitting the use of grades of 5 per cent. in the mountain region.

In November, 1873, the track had been laid to the sugar estate of

Suchiman, about 35 miles from Chimbote, and the location had reached the cañon of Huayna Taquilpon. A preliminary location had been made as far as Tunnel No. 4, but no one had been through the cañon at this time, though a mule road was finished through it within three months of the date named.

I have made these general statements as introductory only, and desire to limit the scope of this paper to the location of the tunnels of the first cañon, although I shall give such details pertaining to the construction of these tunnels as came within my own observation prior to September 1875, when the financial embarrassment of the Peruvian Government caused a suspension of the work.

The rocks of the cañon seem to belong to the carboniferous age, though the strata had been greatly distorted and compacted by subsequent changes, and the coal existing occasionally, in veins from three inches to six feet thick, had been debitumenized and changed almost to graphite. The strata generally dip in the direction of the river, at right angles to the mountain chain, and present some of the most remarkable cases of folding and distortion I have ever seen. The process of change seemed almost a recent one, for nature had never healed the scars. Very little rain falls at this elevation of 3 000 feet above the sea; on the mountain sides water was wanting; there was no vegetation; red, yellow and black were the colors of nature. Rock surfaces were only partially smooth from river washing; the general effect was that of unfinished work recently discontinued.

The Santa is a mountain torrent; its average width about 100 feet, its average depth about 6 feet at low water, and more than 20 feet at high water; its fall through the cañon was about one and one-half per cent., its velocity seven or eight miles per hour. While fool-hardy Chileños have swum across it for large rewards, over fifteen men were drowned in it in less than a year of construction, and I never heard of a single rescue where persons had fallen into it.

The cañon may have been a huge cleft or series of clefts in the rocks, the softer or looser portions washed out to some width, the harder portions remained at first as dams, which backed the river up into lakes in which boulders and coarse gravel, brought by a mountain torrent, became sedimentary deposit; these dams removed, or cut through, the sediment remained in the shape of high gravel flats, through which the river had made an open cut ten to twenty feet deep. These flats, sur-

rounded by high rock walls, were the camping grounds of the cañon, the only tracts nearly level, they furnished the only earthwork, and it was the effort to avail ourselves of the easier work on them which gave us in profile a succession of nearly level grades connected by steep grades through the intervening rock cliffs.

The steep river walls, rising from 200 to 400 feet, seemed vertical, and the mountain sides sloped away from the tops of these walls at slopes often quite too steep for earth to stand. I do not wonder that all the district from kilometers 108 to 111 was embraced on the preliminary study in one tunnel of 2 100 meters (6 890 feet) long.

The location of the tunnels in this cañon became particularly interesting from the fact that in no case was it possible to run the centre line over the top of the tunnel. This gave rise to triangulations more or less extended as the circumstances required. In the triangulations we were frequently compelled to cross the river by means of a rope stretched from bank to bank, carrying a common pulley block from which was suspended a rude seat. In getting about from point to point in location, a gang of men, including at least one sailor, was continually employed making roads and paths. The mule road passed over tops of cliffs, and ran down on to the intermediate gravel flats where practicable. For descent into rock clefts and over the face of cliffs, rope ladders were used, one of 50 feet in length by 12 inches wide; another 104 feet in length by 18 inches in width; these could be carried on donkeys to within a few feet of the spot where they were used. A single rope, lashed to a bar driven into the ground or into a crevice in the rocks, served for examination of very steep slopes, and the "Chileno" chainmen would do some very accurate work on smooth rock slopes of 40 degrees inclination, barefooted, with ropes about their waists, the other end being carried as the chainmen moved, by their fellows on the mule road above. In the locality I have particularly in mind, the mule road had been carried for 200 meters (656 feet) along a glassy rock surface, partly in excavation, partly on little walls supported on horizontal timbers which, in turn, rested on iron bolts driven into holes in the face of the rock.

Plate No. XIX. shows that portion of the department which includes the Santa river, together with a general plan of the line through the tunnel district, with corresponding profile; the contours are 5 metres ($16\frac{4}{10}$ feet) apart, vertically. Plate No. XX. gives details of the most important tunnel triangulations and sections across the river at points opposite Tunnels

Nos. 3 and 7. The triangulations were made as rapidly as possible; the simplest cases were presented by Tunnels Nos, 1, 4, 5, 6 and 8. The detail sketch of triangulation at Tunnel No. 5, illustrates a frequent method of treatment. In general the data desired were the length of a line, as $B\,E$, Tunnel No. 5, connecting two stations at opposite ends of the tunnel, and the angles made by this line with the tangents into and out of the tunnel.

Let us suppose this information given by triangulation and the line

$$BE = C$$

The angle made by tangent at B with $BE = B$

" " " E " $= E$

" of intersection between the two tangents $= B + E = I$.

The perpendicular let fall from point E on to tangent at B, produced, $= b$

we then have 1st, $b = C \sin. B$

and 2d, $b = R \tan. \frac{1}{2} I \sin. I$

$$= R\,(1 - \cos. I)$$

$$\text{Whence } R = \frac{C \sin. B}{1 - \cos. I},$$

in which R is the radius of curve to be used. This expression, representing the relation between the line and angles named, may be taken as an expression for most of the tunnel lines. It was used in this, or in a a modified form, in Tunnels Nos. 1, 2, 3, 5 and 8. Tunnel No. 4 was on a straight line, practically.

The location of Tunnel No. 1 included that of Bridge No. 6, the upper end of which was at the tunnel's mouth. The tunnel began with a curve of 76.3 M. (250 ft.) radius, which extended about half way through it, and ended with a straight line.

It was very important that the tunnel work should be pushed rapidly on account of the difficulty of transportation by mule road through the cañon, and since from the necessities of the case this work would be slower than any other, it would determine the time of the completion of the work on the division. Much attention was paid, therefore, to shortening the tunnels and providing means of entering them between the ends. The adits or galleries in the side of the tunnel took the place of shafts, and were placed, when possible, at points easiest of access, and in such position generally as to divide the work for rapidity of execution.

Each day spent in driving the adit counted twice against the time of completion of the tunnel.

Tunnel No. 3 had been placed on preliminary location at a length of 500 meters (1 640 ft.), its center line would have been about 50 meters (164 ft.) distant from the river wall, as shown on section *E F*, Plate No. 2. Even supposing the use of three adits in the side, these would have averaged 40 meters (= 131 $\frac{2}{10}$ ft.) each in length, and the time of driving the tunnel would have been at least equal to that of driving a tunnel 160 meters long.

A bench, or break in the slope of the river wall, occurred between Tunnels Nos. 3 and 4, with just room enough for the road bed on solid rock in side cut, averaging about 12 meters (39 ft.) deep for 200 meters (656 ft.) in length. Throwing the line on to this bench reduced the length of Tunnel No. 3 to 253 meters (829 $\frac{8}{10}$ ft.), which we were able to divide again by the adit on line *A S* Plate No. 2.

A B was a base line measured on a narrow strip exposed at low water and submerged several feet at high water; *B* to *C* was inaccessible for purposes of measurement; *I* was a point located at the entrance of the adit after the work began, and *P* a point established on line *A S* produced.

The tunnel line was assumed parallel to the base line *A B*, and placed 16 meters (52½ ft.) from it, horizontally, this distance being determined by the position of the point *F*, outside the tunnel.

The cliff through which the tunnel ran was stratified at low inclination, and cleft with distinct and often open joints perpendicular to the plane of stratification. The river for a distance followed one of these joints, the base line was made parallel to them, and it was thought that the straight portion of the tunnel would be more easily driven if made parallel also to the plane of these joints. This plan had its defects in execution, since it favored the opening of a large crack in the roof just above the adit.

The rock wall outside the tunnel was never less than eight meters (26¼ ft.) thick, excepting at the adit, which ran through a soft rock material, filling a large cleft at right angles to the strata and the principal joints.

The strata were bent between Tunnels 1 and 5 into a high arch. Coal seams just under Tunnel No. 1 appeared high over Tunnel No. 3, and ran under Tunnel No. 5 at the river level. The road bed cut the strata

in Tunnel No. 1 at about 45°, and in No. 5 cut them again at about 30° inclination.

At Tunnel No. 6 we were unable to get foot hold on the opposite side of the river without great difficulty. The tunnel line could be produced on top of the tunnel and at *E*, by deflection; the point *F* could be set beyond the tunnel. *A B* was the measured base; *C* and *D* were points carefully selected on the river wall opposite; stations merely, without flagmen. The point *G* was established on the line *G H*, parallel to *F E*, and the location continued from that point.

In getting on to the Pampa Encerrada (pampa is used here in its limited sense) we had been let down by ropes from above, over cliffs 200 feet high.

It was difficult to look into or out from this little space, excepting in a vertical direction; the road to it was improved later, though always bad; several lives were lost during construction, in getting to and from the work. Tunnel No. 7 was, therefore, particularly difficult of access on the west[illegible] or lower end; from this end all the adits were approached along the riv[illegible] edge to adit *C*, and on staging built out over the river from *C* to *d*.

The base line *A B* was measured on a piece of the gravel flats before referred to; beyond *D* both walls of the river became vertical. The points *a*, *b*, *c* and *d* were reached by long ladders from below, up which men climbed to the level of grade, and set iron plugs in holes drilled to receive them. The adit at *b* occurred in another of the clefts filled with softer rock. Station *E* was 90 meters (295 ft.) vertically above grade of tunnel.

On the plan (Plate XX.) full lines represent visual lines, the heavier ones indicating lines of primary triangulation. *D s*, *F p* and *d V* gave direction of the adits at corresponding points. The adit *b s* was $6\frac{37}{100}$ meters (21 ft.) long; the others were shorter, *C* being scarcely more than 3 meters (about 10 ft.). *I* was a point outside of the tunnel at its upper end, and was assumed to be on a tangent from the tunnel. Taking *p* at the intersection of the two lines *F p* and *I K*, simplified the work, since *p* could be set in the tunnel, and the center could be established by tangent deflections until the point of tangency could be set, from which the curve could be run backward. The determination of the point *s* was more difficult; from the intersection angle *k* and assumed radius of curve, 235½ meters (772 $\frac{4}{10}$ feet,), we could locate the point of curvature

q'; the distance $D\,s$ resulted from solution of two triangles $D\,q\,O$ and $D\,s\,O$, where O is the center of the curve $q\,s$.

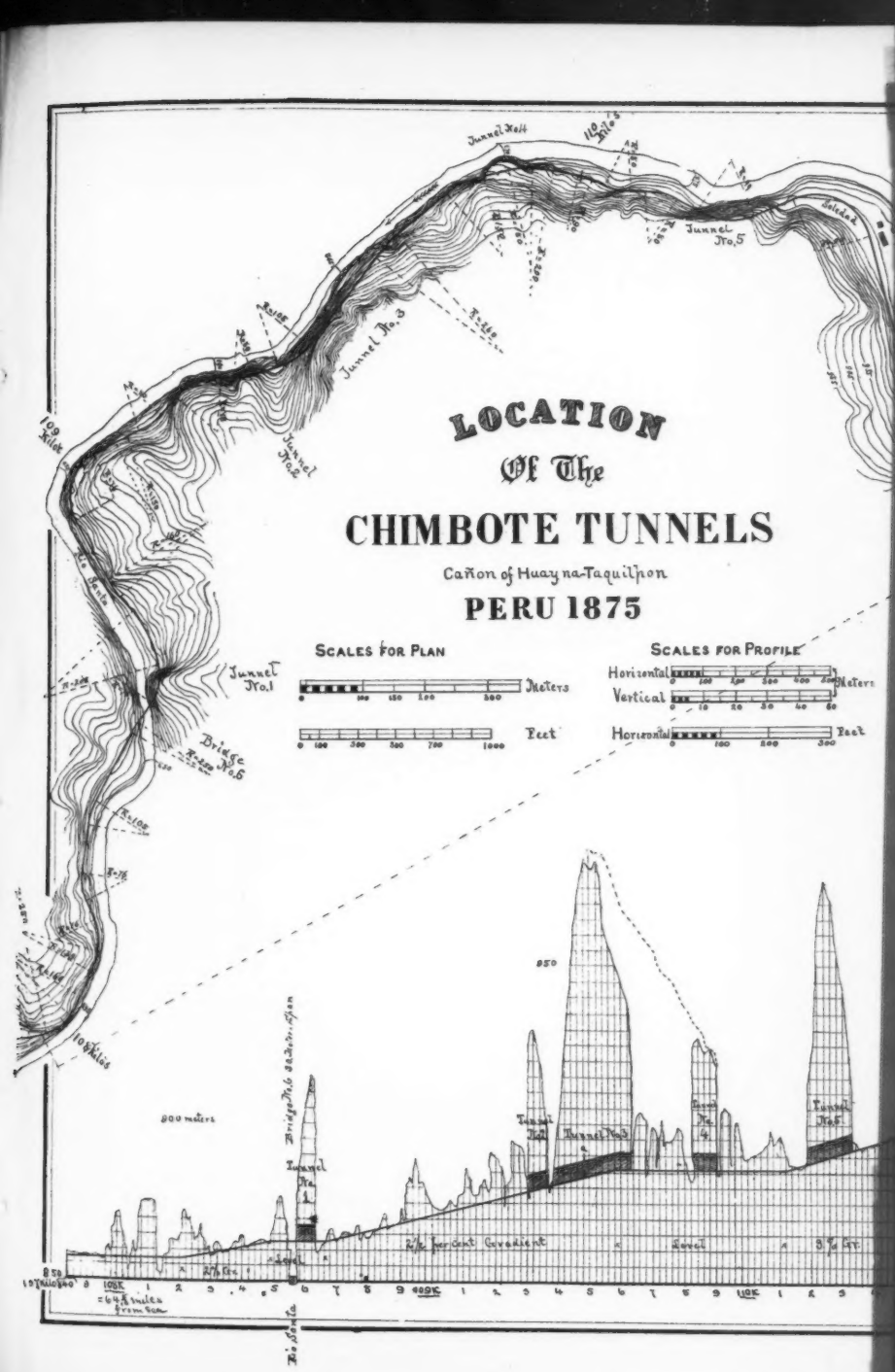
$I\,r$ was taken equal to the difference between the calculated length of $G\,I$ and $G\,r$, $= 17.36$; measured on the ground it was 17.368 meters long; this, though not exactly a proof of the accuracy of the triangulation, served to increase our confidence in the work.

In the location of Tunnel No. 9, which was only regarded as preliminary, the method of Stadia measurement was used for the determination of the chord line referred to in connection with Tunnel No. 5. In definitive location a more extended triangulation would have been made, as in the other instances, preparatory to construction.

In measurement of base lines, Chesterman's steel tapes were used, the measurements being repeated several times in short lengths and between level supports.

The work was done in a climate of almost uniform temperature, and the results obtained justified the means employed. The transit instruments used were generally English, principally of Elliott's make; the vernier read to twenty seconds with good degree of accuracy. All angles were repeated from three to twenty times; where all three angles could be observed, a great number of repetitions was not insisted upon, unless the results were quite unsatisfactory. Seldom, however, were more than five repetitions necessary. A steady breeze blew up the river from ten o'clock in the morning until sunset, and we were often compelled to anticipate its arrival in taking angles from exposed stations.

All observed angles were taken between 30° and 120° where possible. The rule in field work was to establish all stations with greatest security and so that the stake could be seen from the instrument if possible, even at the risk of labor and inconvenience in getting to and from them. At Tunnel No. 3 stakes of 3×4-inch timber were driven intc the earth, or 1½-inch plugs were driven into holes drilled in the rock. A hole bored in these received a No. 8 wire about one foot long, carrying a small red flag at its end; when the instrument was at the station the wire was removed and the instrument set over the hole. Much time was saved in sending men from station to station by use of this method. It will be observed that the angles at Tunnel No. 3 were often small; this seemed unavoidable; there was frequently 60 to 80 meters difference in level between the stations.


LOCATION
Of The
CHIMBOTE TUNNELS
Cañon of Huaynaquilpon
PERU 1875
SCALES FOR PLAN
Meters
Feet
SCALES FOR PROFILE
Horizontal
Vertical
Meters
Horizontal
Feet
Tunnel No.1
Tunnel No.2
Tunnel No.3
Tunnel No.5
Bridge No.6
109 Kilos
10 Kilos
Rio Santa

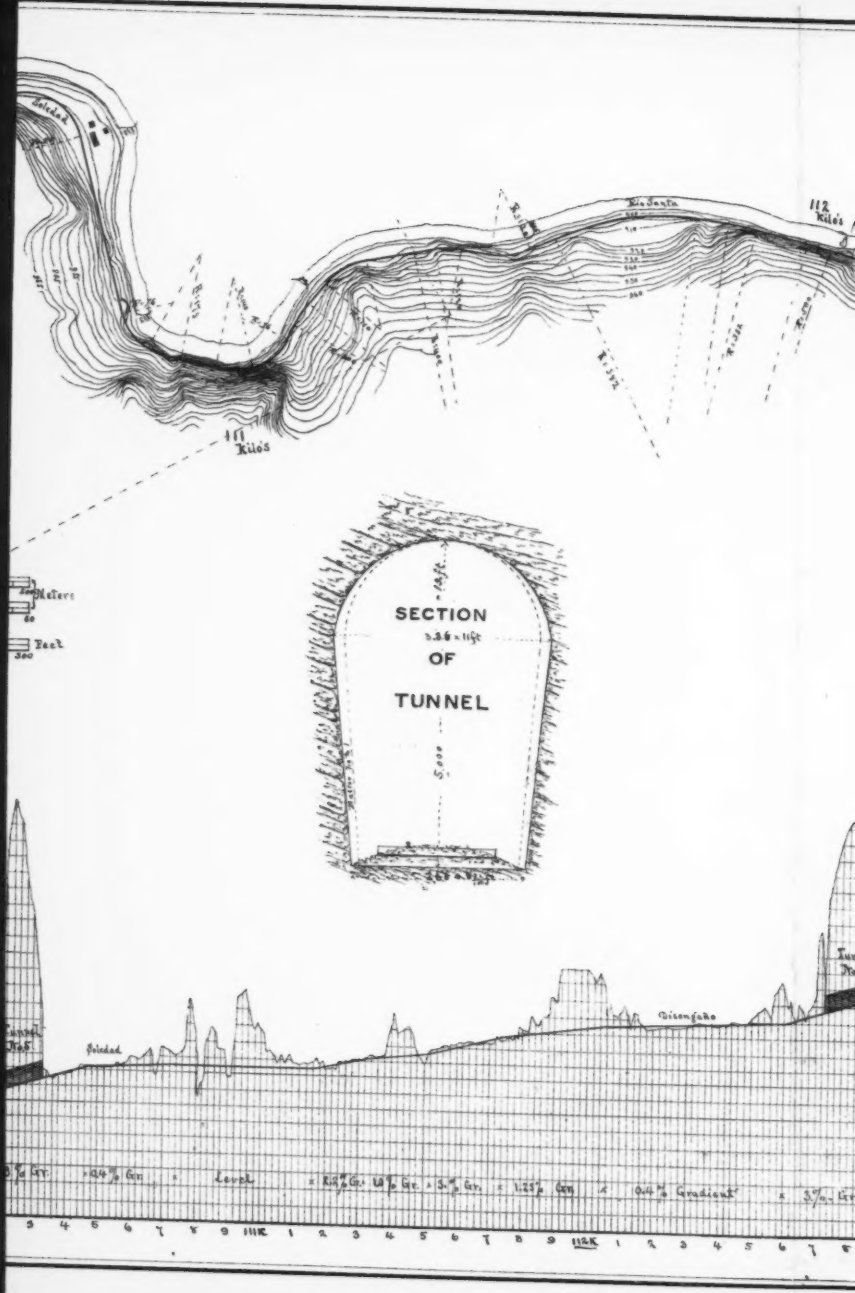

Meters
Feet
SECTION
OF
TUNNEL
Level

PLATE XIX.
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NICHOLSON CHIMBOTE TUNNELS

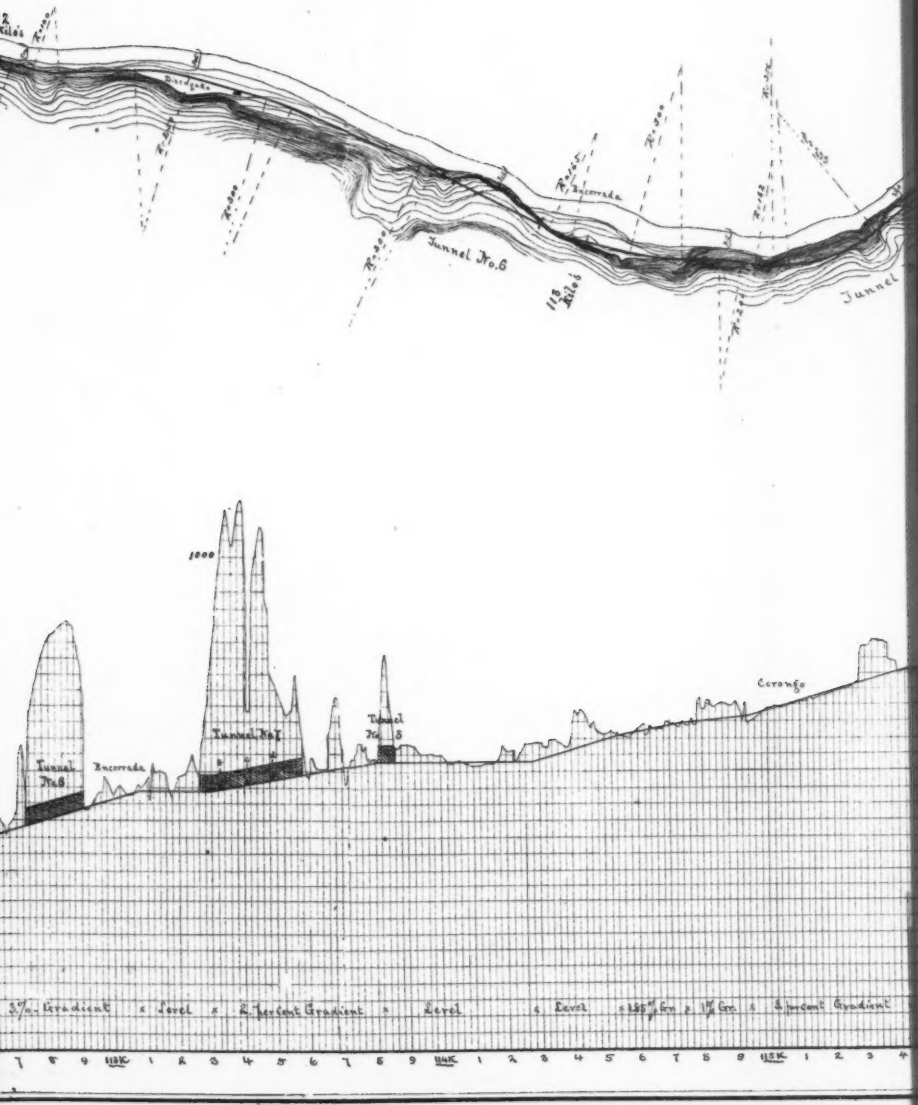

PLATE XIX.
TRANS. AM. SOC. CIV. ENG'RS.
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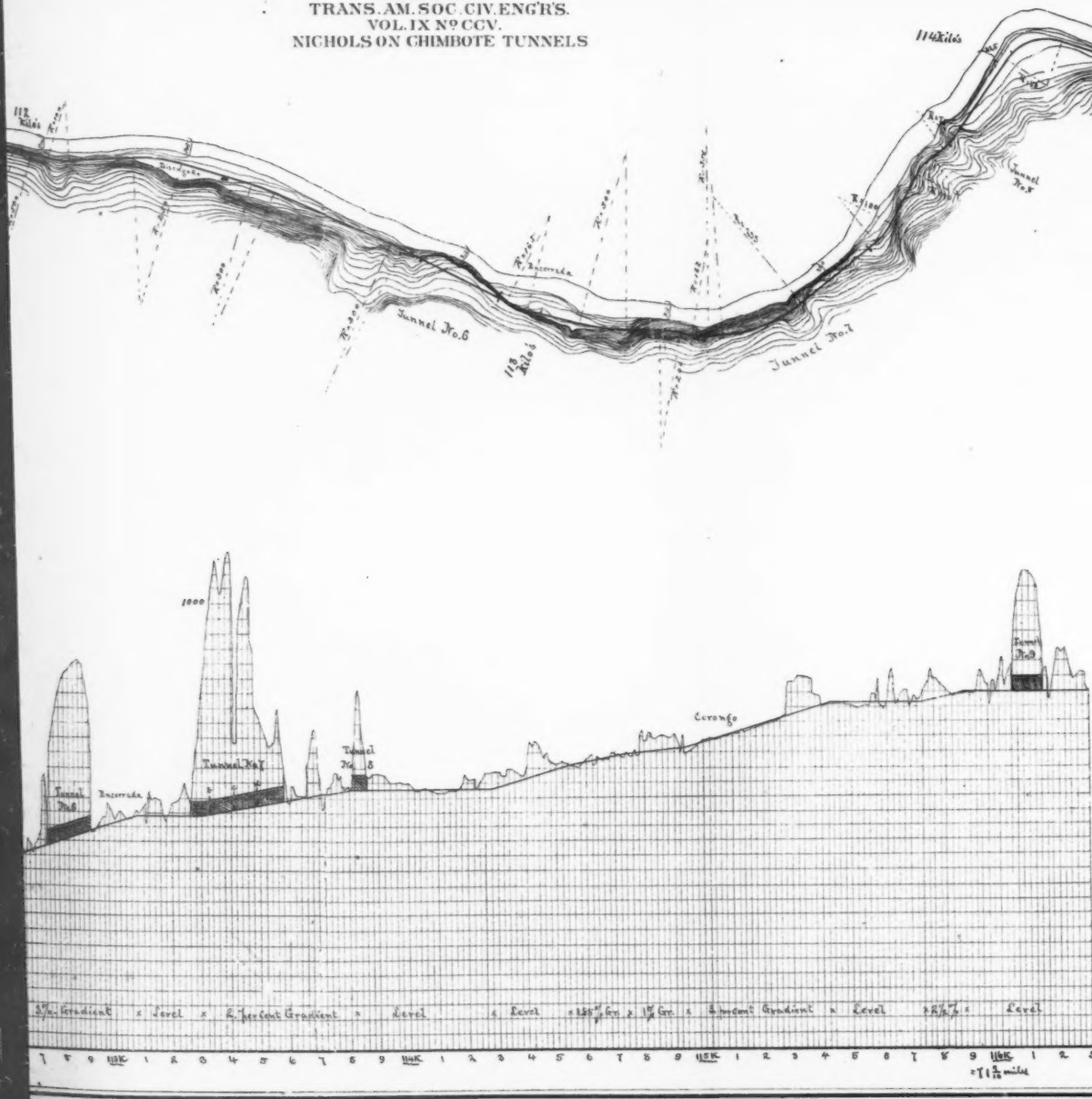
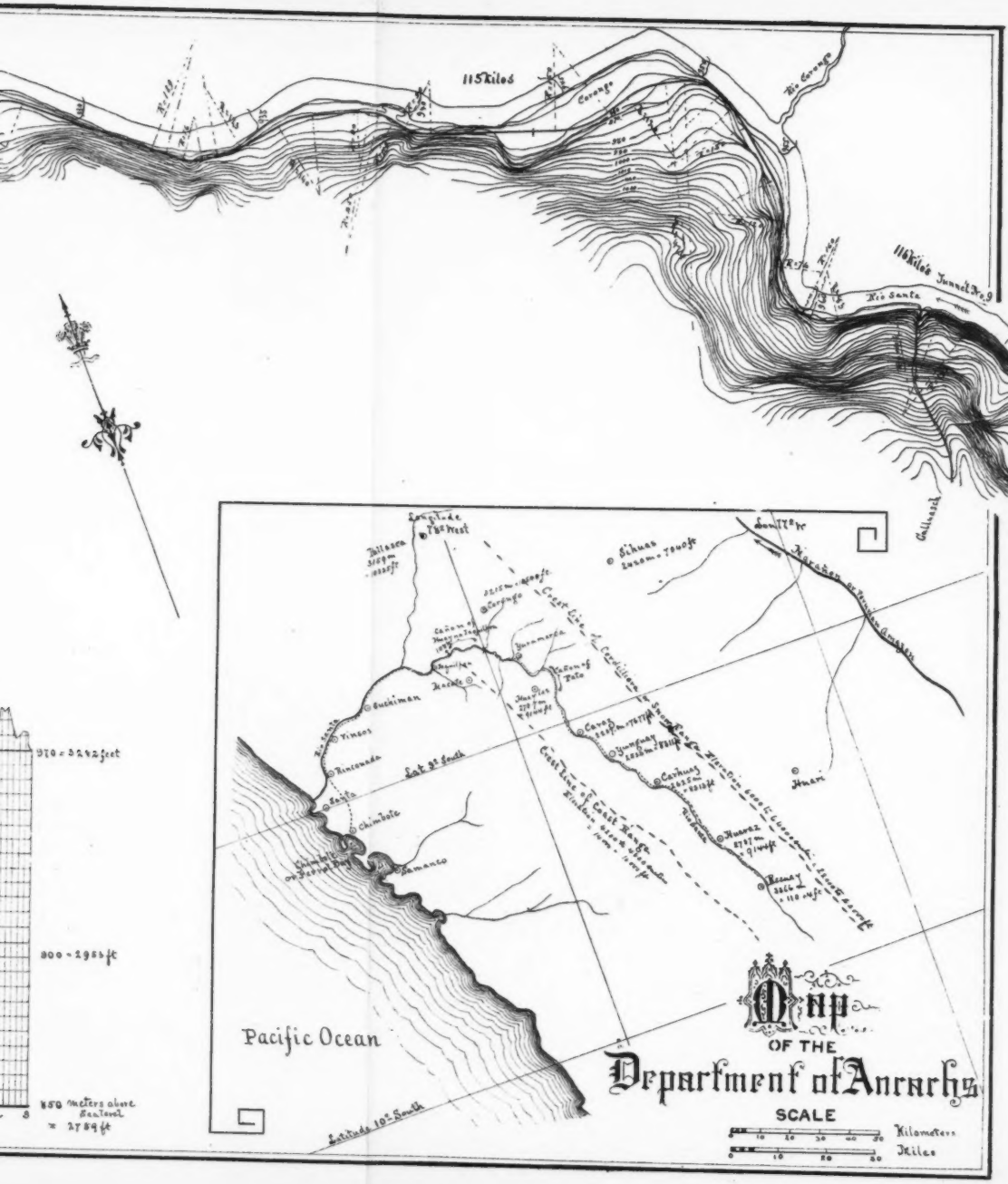

Tunnel No.6
Tunnel No.7
Tunnel No.8
Tunnel No.9
Cerro Negro
Cerro Negro
Cerro Negro


115 Kilos
Corongo
Rio Corongo
115 Kilos Tunnel No. 9
Rio Santa
Callejon
Longitude The West
Tillasca
Corongo
Cahuna
Huancarcancha
Huacaloma
Yuchiman
Yungay
Rinconada
Santa
Chimbote
Samanco
Lat 9° South
Sihuas
Huaraz
Huari
Pacific Ocean
Latitude 10° South
970 = 3242 feet
900 = 2953 ft
850 Meters above Sea Level = 2789 ft
Map
OF THE
Department of Ancrachs
SCALE
Kilometers
Miles

In the summer of 1874 construction began in the cañon; the contract had been sublet to Mr. E. C. DuBois, member of the Society, who, I believe, still holds it; the administration was reorganized, and Mr. Jas. R. Maxwell, member of the Society, was made chief engineer. For more than a year the work was pushed with great energy. The work from kilometer 108 to Tunnel No. 6 was conducted under the immediate charge of Mr. E. T. Scovill, member of the Society, while Mr. D. W. C. Perry had immediate charge from Tunnel No. 6 to the end of the work opened in construction, a point about one-half mile beyond Tunnel No. 9, and including upper heading of Tunnel No. 6. The headings in Tunnels Nos. 1, 2, 3, 4, 5 and 6 had met, and these tunnels were about finished. Tunnel No. 7 was well under way, and with the opening of Tunnel No. 6, more easily approached, when in August, 1875, the work was stopped, as before mentioned.

In Tunnel No. 3, the lines met within two (2) centimeters ($\frac{3}{4}$″) and the results in Tunnels Nos. 1, 2, 4, 5 and 6 were very satisfactory, though not always as accurate as this. The table herewith shows monthly progress at the several headings, as well as best daily progress in all the tunnels. The shorter tunnels were not hurried through, excepting in case of Nos. 1 and 4, where the work had been given to energetic workmen as task work. Tunnel No. 3 was pushed with most energy. This tunnel and No. 2 had been taken together as a sub-contract by one man, a Cornish miner; Tunnels Nos. 6, 7 and 8 constituted another subcontract under a Frenchman who had had much experience in rock work, particularly in Algiers. Between Tunnels Nos. 2 and 3, a rope ladder led vertically 50 feet from the tunnels to a small spot 100 feet square, and not level by any means, where houses were built, tents put up and a store established, and where the workmen employed in the two tunnels lived; a steep, zig-zag road about 2 feet wide led to the main mule road about 200 feet in elevation above the camp.

The tunnel section as stipulated for in the Government specifications, was 5 meters ($16\frac{4}{10}$ feet) in height above sub-grade, 3 meters ($9\frac{8}{10}\frac{4}{6}$ feet) wide at springing of arch, and about $2\frac{1}{2}$ meters (= $8\frac{1}{2}$ feet) wide at subgrade. There was nothing in the specifications to prevent use of this section on curves of 61 meters (200 feet) radius. Radii of less than 76.30 meters (250 feet) were never used on the Division, and the width of tunnel section at springing of arch was made 3.36 meters (= 11 feet) in practice, the sides sloping at slope of $\frac{1}{10}$ to 1. The area of tunnel sec-

tion was about 15 square meters, 5 of which at top were taken out in heading, and the remaining 10 meters in bottom were removed in enlargement.

About 600 "Chilenos" and Peruvians were employed in the cañon. The Peruvians were mainly inexperienced in railway or mining work, and were valuable chiefly as laborers until they became accustomed to use of hammer and drill. They received about 80 cents per day. The "Chilenos" were more energetic and hardy, and hence, perhaps, more troublesome. From long experience in the mines of Chili and in the tunnels of the Lima and Oroya line in Peru, they had become valuable tunnel men. They generally received $2 to $2.50 per day for day work, but did much better at task work, and were allowed $50 for labor of driving heading and $40 for labor of enlargement per lineal meter of tunnel. Most of the rock was very hard to drill, frequently using up 12 to 20 good points in a hole 18 inches deep. The limited width of the tunnels prevented use of more than two gangs of drillers in heading, who drilled 6 holes each in day turn and 7 in night turn, each hole being 18 inches deep; the holes were drilled near the outside of the section, the middle portion being broken out by wedges. It is perhaps impossible to make a valuable comparison, but I should think the progress and the cost of work in these narrow tunnels were 25 per cent. less and greater respectively than on the Lima and Oroya line when the tunnels were 50 per cent. wider for a broad gauge road.

The use of pure nitro-glycerine was not permitted by the Government. Dynamite was generally used in the heading holes, as powder seemed almost useless for driving the heading and its use was studiously avoided for this purpose.

The work was so managed as to ensure economical results, very small profits inured to the sub-contractors, while by contract work many of the evils incident to day labor were avoided. Materials of every kind used in camp and on the work had to be brought from the coast. The distance to end of track was about 50 miles; transportation over this distance was effected by mules, and the cost of carrying a single cargo of 250 pounds was about $10.

As has been stated, the cost of labor on heading was about $10 per cubic meter, and on enlargement the cost of labor was about $4 per cubic meter. The cost of explosives, lights, tools, blacksmith's work, and wages to foreman would double these figures.

Name of Tunnel.	Work Begun.	1874.							
		September.		October.		November.		December.	
		Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.
No. 1, upper end	Nov. 1874.	....	....	....	....	11	36.1	10	32.8
" 2, " heading	" "	....	....	....	....	14	45.9	5	16.4
" 3, " "	Oct. "	....	....	10	32.8	14	45.9	11	36.1
" 3, adit upper heading	" "	....	....	5	16.4	8	26.2	10	32.8
" " lower "	" "	....	....	3	9.8	7	22.9	13	42.6
" 3, lower "	" "	....	....	13	42.6	16	52.5	14	45.9
" 4, upper "	Sept. "	4	13.1	12	39.4	17	55.8	....	....
" 4, lower "	" "	7	22.9	14	45.9	16	52.5	....	....
" 5, upper "	" "	10	32.8	16	52.5	17	55.8	18	59.0
" 5, lower "	Nov. "	....	....	4	13.1	6	19.7	13	42.6
" 6, upper "	Oct. "	....	....	8	26.2	12	39.4	9	29.5
" 6, lower "	Nov. "	....	....	....	....	8	26.2	11	36.1
" 7, " "	Mch. 31st, 1875.	....	....	....	....	....	....	....	....
" 7, adit b, lower heading	April, "	....	....	....	....	....	....	....	....
" 7, " upper "	" "	....	....	....	....	....	....	....	....
" 7, " c, lower "	" "	....	....	....	....	....	....	....	....
" 7, " upper "	" "	....	....	....	....	....	....	....	....
" 7, " d, lower "	May, "	....	....	....	....	....	....	....	....
" 7, " upper "	" "	....	....	....	....	....	....	....	....
" 7, upper heading	April, "	....	....	....	....	....	....	....	....
" 8, lower "	" "	....	....	....	....	....	....	....	....
" 8, upper "	Mch. 31st, "	....	....	....	....	....	....	....	....
Total		21	68.8	85	278.7	146	478.9	114	373.8

Monthly Progress of Work on Chimbote Tunnels, Peru, 1874 and 1875.

		1875.																Be
December.		January.		February.		March.		April.		May.		June.		July.		August.		P
Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	Meters.	Feet.	
10	32.8	16	52.5	5	16.4	8	26.2	....	....	....	....	....	....	....	....	....	....	0.
5	16.4	9	29.5	4	13.1	12	39.4	5	16.4	6	19.7	8	26.2	....	....	....	....	0.
11	36.1	12	39.4	10	32.8	10	32.8	6	19.7	5	16.4	5	16.4	....	....	....	....	..
10	32.8	10	32.8	10	32.8	7	23.0	5	16.4	5	16.4	4	13.1	....	....	....	....	0.
13	42.6	14	45.9	5	16.4	....	....	....	....	....	....	....	...	....	...	....	....	0.
14	45.9	13	42.6	8	26.2	....	....	....	....	....	....	....	....	....	....	....	...	..
....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	0.
....	....	....	....	....	....	....	....	...	....	....	....	....	....	....	....	....	....	..
18	59.0	18	59.0	12	39.4	....	....	....	....	....	....	...	....	....	....	....	....	0.
13	42.6	14	45.9	12	39.4	....	....	....	....	....	....	....	....	....	....	....	....	..
9	29.5	7	23.0	2	6.6	7	23.0	8	26.2	8	26.2	5	16.4	15	49.2	12	39.4	0.
11	36.1	7	23.0	2	6.6	9	29.5	5	16.4	9	29.5	9	29.5	15	49.2	12	39.4	..
....	....	....	....	....	....	...	....	4	13.1	8	26.2	6	19.7	8	26.2	3	9.8	0.
..	...	...	....	....	....	....	....	2	6.6	5	16.4	5	16.4	4	13.1	8	26.2	..
....	....	....	....	...	....	....	....	2	6.6	3	9.8	9	29.5	5	16.4	5	16.4	..
....	....	....	....	....	....	....	....	2	6.6	2	6.6	3	9.8	5	16.4	4	13.1	..
....	....	....	....	....	....	....	....	7	22.9	8	26.2	6	19.7	7	23.0	6	19.7	..
....	....	....	....	....	....	....	....	....	....	4	13.1	7	23.0	6	19.7	4	13.1	..
....	....	....	....	....	....	....	....	....	....	4	13.1	7	23.0	8	26.2	3	9.8	..
....	....	...	....	....	....	....	....	3	9.8	5	16.4	7	23.0	8	26.2	5	16.4	..
...	....	....	....	....	....	....	....	4	13.1	4	13.1	6	19.7	10	32.8	....	....	0.
....	....	....	....	....	....	....	....	6	19.7	6	19.7	6	19.7	8	26.2	....	....	..
114	373.8	120	393.6	70	229.7	53	173.9	59	193.5	82	268.8	93	305.1	99	324.6	62	203.3	

Best Daily Progress.		Work on Heading Completed.	Length of Tunnel.		Remarks.
Meters.	Feet.		Meters.	Feet.	
0.97	3.2	April 20, 1875.	50	164.0	22 hours work daily. Hard and compact material; solid but not hard to drill.
0.73	2.4	June 24th.	63	206.6	22 hours daily.
....	....	" 12th.			22 hours work daily.
0.80	2.6				Hard and compact at ends.
0.80	2.6	Feb. 20th.	253	829.9	Hard to drill and broken, with seams in middle.
....	....				
0.71	2.3	Dec. 1st.	70	229.6	22 hours work daily. Hard to drill but loose in places.
....	....				
0.83	2.7	Feb. 19th.	140	459.2	Hard and solid lower end, loose in upper end. 22 hours work daily.
....	....				
0.72	2.4	Aug. 19th.	180	590.4	Hard and compact at ends; loose in centre. 22 hours work daily.
....	....				
0.28	0.9				Work stopped August 25th.
....	....				No. 7 Tunnel worked only 10 hours per day.
....	....				
....	....		319	1046.3	Very hard and compact material, and difficult of access.
....	....				
....	....				
....	....				
....	....				
0.34	1.1	July 31st.	50	164.0	Very hard and compact.
....	....				Rock. Worked 10 hours per day.
			1 125	3690.0	

The cost of Tunnels Nos. 2 and 3 was about $12 per cubic meter = $10 per cubic yard.

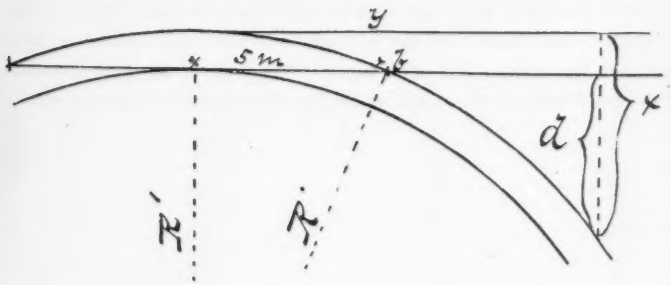
The cost of Tunnels Nos. 6 and 7 was about $14 per meter = $11.70 per cubic yard.

I believe as high as $105 per linear meter was paid on Tunnels 6 and 7 for labor in excavation.

When the rock was loose the roof would be quite unreliable, and provisions had been made for some lining at ends of Tunnels 3 and 4, and may prove necessary at other points.

Line was generally given in tunnels at every few meters of progress. The method of fixing plugs in the roof was not used excepting for important points requiring to be set before bottom was taken out. Since the enlargement followed the heading within about ten to twenty meters it was always possible to keep plugs in the tunnel bottom over which instruments could be easily set. When plugs could be conveniently set in the roof at ten meters apart a table of deflections was sometimes given to direct the workmen until instruments could again be brought into the tunnel.

Two examples are given below of these tables for curves of 120.8 meters, and 301 meters radius. The deflections are given for each meter of the chord produced, and were obtained by deducting difference of radii of circumscribed and inscribed circles from the values of ordinates to the circle corresponding in position to the desired deflections.



R = 120.8 Meters.	R' = 120.696 Meters.	R = 301.574 Meters.	R' = 301.429 Meters.
Distance on chord produced from b.	Values of deflection or offset = d.	Distance on chord produced from b.	Deflection or offset = d.
0 meters.	0.0 meters.	0 meters.	0.0 meters.
1 "	0.041 "	1 "	0.013 "
2 "	0.099 "	2 "	0.039 "
3 "	0.162 "	3 "	0.065 "
4 "	0.231 "	4 "	0.093 "
5 "	0.312 "	5 "	0.124 "
6 "	0.402 "	6 "	0.159 "
7 "	0.498 "	7 "	0.194 "
8 "	0.602 "	8 "	0.240 "
9 "	0.714 "	9 "	0.284 "
10 "	0.835 "	10 "	0.333 "

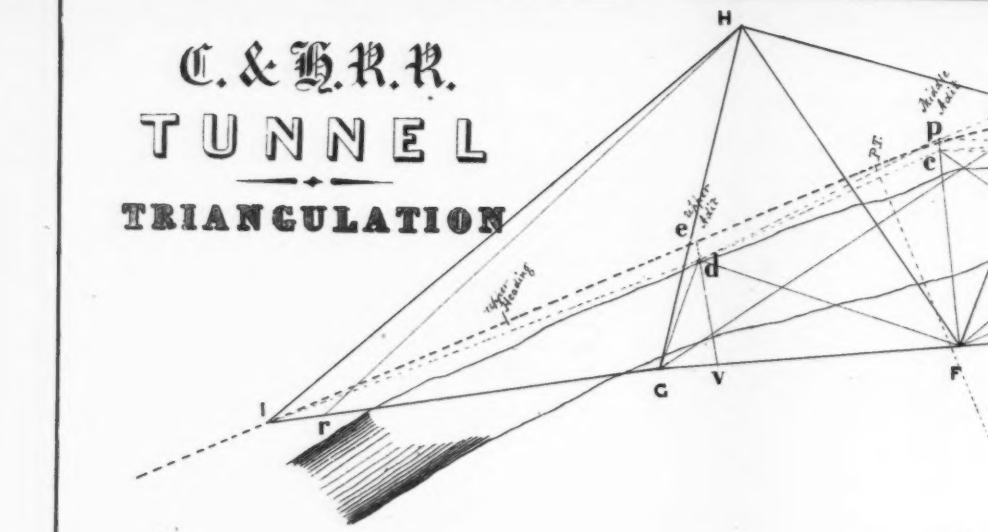
The comparative ease with which the work in the long tunnel represented on the preliminary study at 2 100 meters had been reduced in the five small tunnels by use of sharp curves, narrow road-bed, and close examination of the cañon, leads me to believe that the cañon of Pato, often referred to as *the* cañon of the Santa, would lose many of its terrors on closer examination. A glimpse once obtained into the lower end of this cañon, with some knowledge of the district surrounding it, convinces me that the difficulties would not be greater per mile than those of the lower cañon already passed.

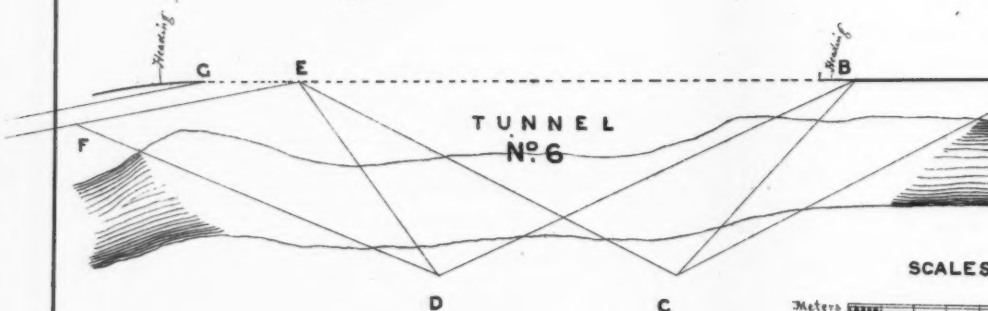
Peru has fallen upon unfortunate experience of late. Whether the Chimbote and Huaraz Railway will ever be completed is, indeed, a question of the future, but it is certain that none of all her projected railways deserves more careful attention, or possesses greater interest.

C. & H.R.R.

TUNNEL

TRIANGULATION

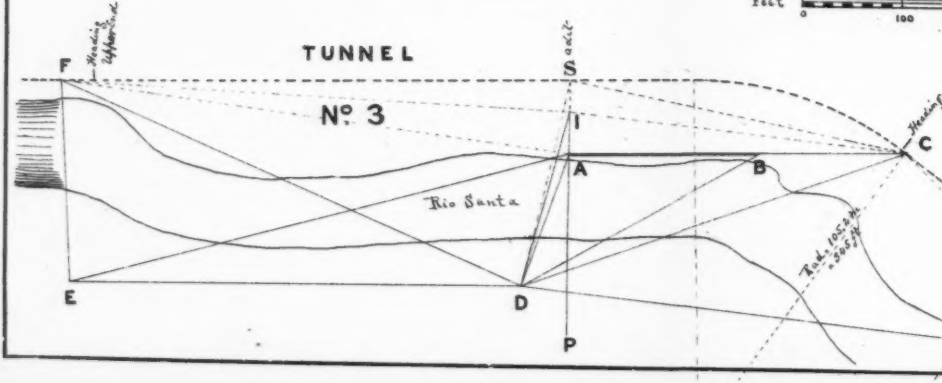




SCALES

Meters

Feet



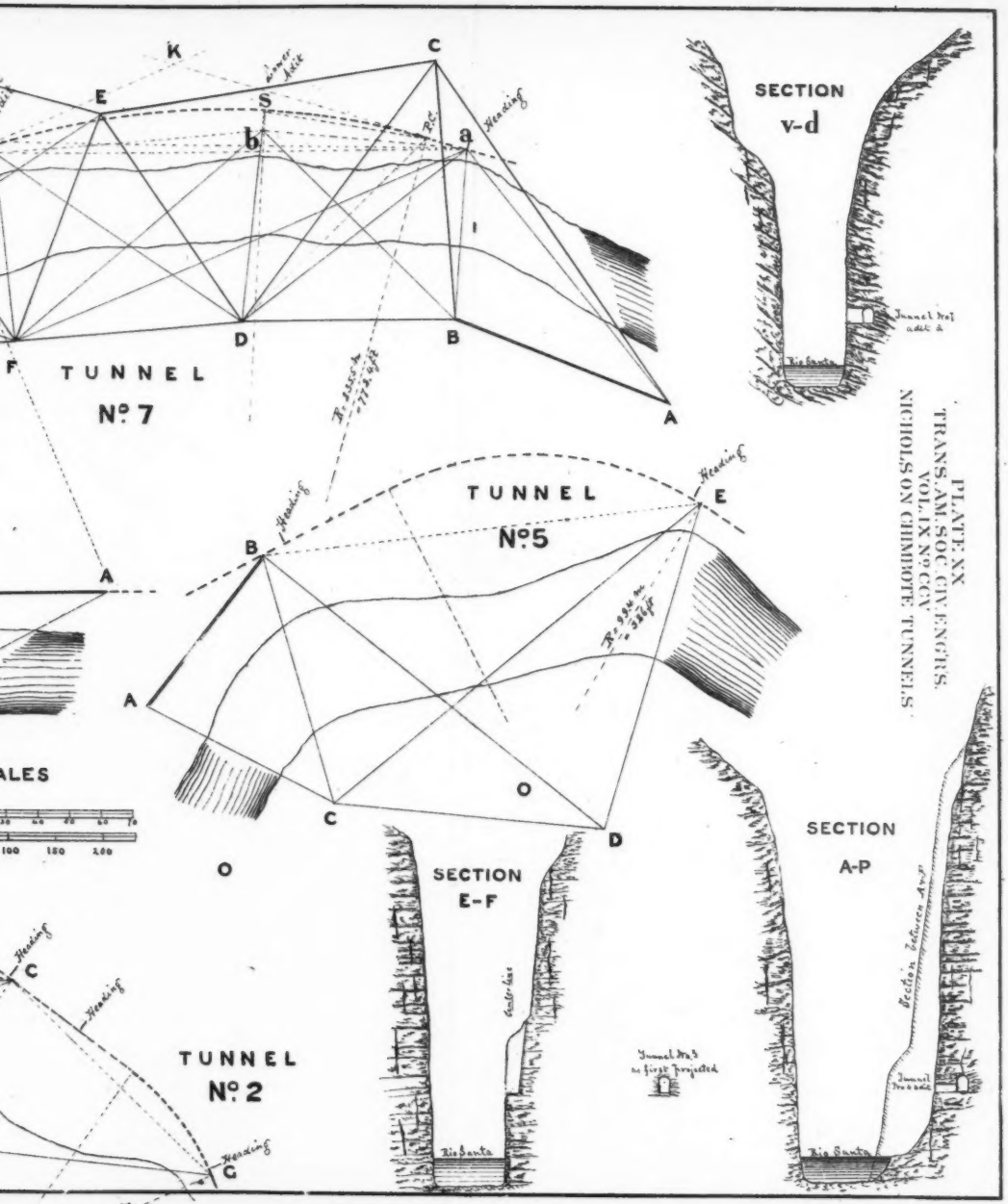


PLATE XX
TRANS. AM. SOC. CIV. ENGRS.
VOL. IX. No. CCV.
NICHOLSON CHIMBOTE TUNNELS.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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CCVI.

(Vol. IX.—October, 1880.)

PRACTICAL CONSEQUENCES OF VARIATION OF THE WET SECTION OF RIVERS UNDER GENERAL AND SPECIAL CONDITIONS.

By ROBERT E. MCMATH, Member of the Society.

READ AT THE TWELFTH ANNUAL CONVENTION, MAY 25TH, 1880.

The particular theme in view requires the consideration of effective cross-section, and the influence which change of the several elements that enter into it has upon the flow of water, the maintenance of channels, and the many practical issues which depend upon them.

The idea of effective cross-section is an abstraction or a generalization which gathers into one expression all the facts and influences affecting the discharge at a given point. For a given stream at a given time the product of area and velocity is a constant; therefore their variations must be simultaneous, opposite in sign, and in equal ratio. But it is soon discovered that equal areas are not equivalent, therefore actual areas do not meet the necessities of logic, and resort is made to the ideal section of equated value, which is called the effective cross-section.

The elements of actual area are few and simple, depth and width The elements of effective section are many and complicated, embracing in addition to depth and width, trace or outline of banks, form of section, contour of bed above and below, character of the bed and banks in respect to roughness and regularity ; also the tenacity or mobility of the material composing the bed and banks. These are permanent elements at a given locality, and there are also temporary elements, such as air currents over the surface, and the presence of accidental obstructions dividing the flowing waters into separate streams, which, while divided, are retarded by frictions, and when reunited, produce eddies and cross-currents.

The naming of these elements (the list is not exhaustive) is sufficient to show that the subject does not lie within the range of precise or mathematical investigation. The only possible mode of general treatment is to assume the simple conditions, examine their effect singly, and then in combinations approaching actual cases.

First assume a stream with straight and parallel banks, a rectangular or other symmetrical cross-section, and a smooth unyielding bed of homogeneous material. Obviously these are the conditions suited to an uniform effective section, and equal areas of similar figure taken at different parts of the section will have equivalent value, except as affected by variation of velocity in the horizontal and vertical directions. In this case these curves may be considered regular, the former symmetric about the axis of the section. Increase or decrease of volume will produce no radical change in any of the conditions.

Next, suppose the trace a well defined curve, other conditions as before. The equivalence of equal and similar areas, symmetrically situated with reference to an axial line, disappears as a result of the tangential force, and the horizontal curve of velocity is no longer symmetrical. If the cross-section be released from the arbitrary condition of symmetry, the natural arrangement will be a shelving shore on the convex side, and an abrupt bank on the concave, whence a greater disturbance of the equivalence of areas will follow, and additional distortion to the curve of velocities. Variation of volume will now overthrow the condition of parallelism, for the change in the form of cross-section just allowed will be gradual from the straight trace and symmetrical section, to the point of greatest curvature and concentration of current. Therefore, the outline on the convex side at a low stage cannot be parallel to that at a

higher, and if the condition of parallelism of the two banks, or uniform width, was imposed at a high stage, it will be broken at a low, and *vice versa*.

Under the conditions now supposed, another phenomenon will manifest itself with change of volume, which may be defined as vagation of the axis of greatest velocity. The gentle slope of the convex shore occasions considerable change of width as the waters rise and fall, hence the centre of figure will travel toward the concave bank during decline, and toward the convex side during the rise. If the hydraulic axis of the stream be taken at several stages, by multiplying each element of cross-section by the velocity of flow at its position, and equating the products about a line, which is equivalent in determining the axis of the discharge, the gyration of the hydraulic axis with change of stage becomes evident, also that the range of its movement is greater or less than that of centre of figure, as the axis of greatest velocity falls on one side or the other of that centre. This in part follows from the augmented effectiveness of the areas as the depth of flow is increased.

Effectiveness, other conditions being the same, varies as the square roots of the depths; therefore the rate of variation in effectiveness is more rapid in the shoal than in the deep parts of the section. When the waters rise, the velocities in different parts of the section approach uniformity, but so long as the view is restricted to the single idea of form of section, it does not appear that they can reach, much less pass, that limit into the condition of greatest velocity not coinciding with greatest depth, which it is well known often occurs.

Considering that the hydraulic axis represents the actual stream, it will appear that any change in its position is virtually a change in the trace of the whole stream, which will indirectly lead to modified form of section, and directly cause increase or decrease of velocity, by the variation in length of trace which is inseparably connected with change of position. The relative velocities in the several parts of the section will also be affected by the change of curvature or trace. In Fig. 1, Plate XXI., the broken line represents the trace of the hydraulic axis at a low stage, the dotted line the more direct trace at high water.

The variations of the hydraulic axis during a rise are almost invariably in the sense of straightening, whence the course of the current is shorter and the slope greater at high stage than at low. The actual velocity along the concave bank may not be diminished, but the increase on the

opposite is often great enough to render it relatively less, whereas it was the greater.

It is, perhaps, necessary to say that this reasoning does not apply with equal force to every curve, or to all parts of the same curve, at a given time, but to each in its own time and order. The law by which currents incline toward concave banks always holds good. But the action of this and other laws varies in position, as the volume is increased or diminished, by a law also absolute.

If in pursuing the study, the remaining condition of unyielding bed and banks be released, the stream assumes the natural type of a river traversing an alluvial region. The variations of the flow will exert an influence upon the bed and banks hostile to their stability. But if the stream be a silt bearer and the bed extremely unstable, the cross-section will strive to adapt itself to the changing conditions of flow, with a readiness of adjustment which tempts the observer to forget that the power of adaptation is subject to limits of extent and time. Without such limits the condition of uniform motion would be attained.

By this partial adjustment a fill will occur on the convex and scour along the concave side during the declining and low stages, and the straightened, shortened trace, during the rising and flood stage will, to a limited extent, develop a tendency to scour the convex, and in some cases to fill the concave side. The first of these statements will probably not be questioned. Owing to the progressive erosion of bends, and the permanent extension of points, the scour of the convex side in time of flood is concealed, except as obscurely indicated by the flattened profile of shore bars and their frequent separation from the point by high water chutes, because the eroded material is replaced during the decline. The limited filling on the concave side quite often remains in fragmentary patches of mud, visible when the water falls. Citation of particular facts, or the testimony of individual observers, is not consistent with the purpose of this paper.

The fact that the extremes of volume have a contrary influence upon the cross-section involves necessarily a transition period of no action, and consideration of the changed trace of the hydraulic axis determines that the point of action, be it scour or fill, will vary with every change of stage. Therefore all parts of an extended bend will never be affected in the same sense at the same time; but in a stream having well defined succession of high and low stage all parts will in turn be subject to action

in proportion to the duration of particular stages. If the changes of stage are always progressive from one extreme to the other, the curves will be regular. Interruption of a progressive change, or irregularity of period, occasions confusion of curvature, whence the outline of a bend usually presents a succession of intersecting curves. In general terms, the condition of scour will prevail where the hydraulic axis is convergent to the bank, fill where the axis diverges from the bank, and no action where axis and bank are parallel. It is therefore impossible to trace in detail the action of the general law that change of form, position, and effectiveness of section attends variation of volume in streams which form their own beds. But it is possible to draw the following conclusions, having their application to the curved parts of silt-bearing streams :

1st. High water changes within the bed proper are not, as a rule, lasting, being commonly more than compensated by the opposite low or falling stage changes. They are not, therefore, hostile to the permanent retention of the stream within fixed limits.

2d. Low stage changes can only be controlled by preventing erosion of the concave bank.

3d. If erosion of the concave bank is checked the force of the stream will be expended in widening and deepening the low-water section, chiefly the latter. No protection of banks can therefore be permanent unless the foundations extend to the depth of this probable scour, or protection be given to the bottom. The necessity for deep foundations will generally be limited to the upstream part of the bends where the low-water action is most intense.

4th. The slackening of the currents at high water along the upstream part of the bends renders the artificial protection of the banks above the mid-stage level a matter of secondary importance. The acceleration of the same currents near the lower limits of the bend calls for the effectual protection of the higher parts of the bank, but the foundations will be comparatively secure.

5th. No works designed with reference to the conditions existing at one extreme stage can be operative at the opposite extreme.

6th. The greater the range of volume between the flood and low stages the greater will the extent of change in the cross-section be, and the more serious the attack upon the stability of works erected to establish a given condition.

7th. The opposite effect upon the cross-section, due to extremes of

stage, is one of the conservative compensating principles by which the essential stability of natural arrangements is made possible in the midst of constant change.

In theory, regulation of width seems to promise an equalization of depths and velocities, whence it may be argued that the silt transporting power would be constant, and erosion and deposit must cease. This overlooks the influence of trace and its variations upon the direction and distribution of currents, and so applies generally principles which are good only when limited to straight channels. The equalizations of theory extend only to mean depths and velocities, leaving the variations within the cross-section out of view. Clearly the water which flows along the concave bank in the bend, will pass along the shoal margin of the point below with very different velocity and transporting power, whence it must be that the individual parts of the silt burden undergo continual exchange, even if the total quantity remain constant, therefore fills and erosions are necessary.

Hitherto the changes affecting different parts of the same cross-section have been considered. The next study will be the more general changes which cause difference in effectiveness of sections more or less closely related to each other by position. The imposition of arbitrary conditions is no longer necessary, and the study will be of things as they are, referring to experience and observation for verification of statements necessary to the argument.

In a series of cross-sections extending to the mean level of the principal banks, if the variation of width is gradual but considerable, the location of the section of greatest efficiency will be variable with the volume, and the locations of related sections of greatest and least efficiency will interchange as the stream rises above or falls below a particular stage. The velocity at the narrowest section will be a maximum when the flood is at the highest, and a minimum when the volume is least. At the widest section the converse is true in a limited sense.

Let Fig. 2 (Plate XXI.), represent the section of least width in the series, whether located in a bend or not is immaterial, except that in a bend the form of section is definite and like the figure.

Fig. 3 represents the section of greatest width in the series. The shaded lines represent the bottom of the sections at a particular stage when the areas of the two sections are equal. The upper and lower

horizontal lines are the water surface at the extreme stages, and the intermediate line is the stage corresponding to equal areas.

It will be necessary to remember, since by the general formula $V = C\sqrt{r\,s}$, that in similar channels, other conditions being equal, the velocities are as the square roots of the hydraulic mean depths; therefore the velocity at any particular point in a cross-section will vary nearly as the square root of the depth at that point, when the river rises or falls, and the mean velocities of the sections as a whole will vary nearly as the square roots of the mean depths; for in large rivers the difference between mean and hydraulic mean depth is inconsiderable. A quantity, which varies as the square roots of an arithmetical series, will have a rate of variation rapid at the beginning of the series but becoming smaller as the terms of the series increase, like the ordinates of the curve whose equation is $y^2 = x$.

Since the mean depths at the stage of equal areas are unequal, being in the case inversely as the widths, the equal areas of the two sections will not with a given slope pass an equal volume. In order that the volume, which must be the same at the two sections, may be discharged, the mean velocity at the shoal section must be made equal to that of the deeper one by a steeper local slope. If the river falls, the areas will diminish unequally, the wide section losing the most. The diminution of depth will also affect the hydraulic capacity of the shoal section much more than the deeper. The now unequal areas have a still greater difference of effectiveness than before, consequently the velocity and slope at the shoal section must each undergo a very considerable increase, and at the deep section a slight decrease. Carried further, the shoal becomes the location of riffles or even rapids, and the deep places pools without apparent motion at low water.

Returning to the stage of equal area, and supposing the river to rise, the increase of area and effectiveness will be more rapid at the wider section, wherefore the capacity for discharge will speedily overtake that of the narrower, and the mean velocities will be equal with equal slope. If the rise continues, the wide section becomes the more effective, and the greater velocity and local slope pass to the narrow section. The difference of velocity at the sections will increase as the height of the flood increases, the current in the narrow parts of the river becoming stronger, and in the wider parts comparatively slack. But the range of slope and

velocity between extremes will be much greater at the wide section than at the narrow.

These changes with the stage are familiar facts to every one acquainted with streams of stable beds. In a silt-bearing stream, traversing a bed whose material is readily moved by currents, the influence of the varying effectiveness is divided between changes at the bottom and the exchanges of velocity described above. The effort being to equalize the effectiveness of cross-sections, and so maintain uniform motion, quickening of velocity is in such streams checked by enlargement of area through erosion, and slackening by diminished area through deposit. The adjustment, in the interest of equalized section and uniform velocity, will be the closer as the mobility of the material composing the bed increases.

It is, therefore, demonstrated that in silt-bearing streams, traversing beds composed of their own deposits, a building up of bars and shoals in the wide places, and a scour in the narrower, must occur in time of flood, and a waste or scour of the shoals, with fill in some part of the pool below, must occur as the river falls to a low stage.

Reproducing the sections as Figs. 4 and 5, the dotted lines are added to represent the probable sections at the extreme low stage, and the broken lines the same at the culmination of the flood. The contrary effect at the extremes involves a transition period of no action upon the bed. This period plainly must coincide with the nearly equal velocity which accompanies equal efficiency of sections. This equality occurs at a particular stage as the river rises, and again as it falls. Is this particular stage the same in each case, and at all rises and falls, or is it variable?

Equivalence of areas is the test, and exists when the area at the wide section is somewhat greater than at the narrow, the occurrence of these conditions can be determined approximately, and the question answered. If the river be supposed to have fallen from the intermediate stage of equal area to low water, and the sections to be changed from the shaded to the dotted outline, a line to cut off equal areas of the new sections must be drawn considerably below the intermediate line of former equal areas. As the increase of velocity at the shoal section in the case of unchanging bed was much greater than the decrease at the deep one, in the present case section changes replace velocity change in part, and must measurably be in the same proportion; therefore, the scour at the

wide section must be greater than the fill at the narrow, and the line cutting off equal areas will be lowered. The lines cutting off equivalent areas will lie slightly higher on the wide, and lower on the narrow section, than the line of equal areas. Therefore, the closer the adjustment to equal effectiveness at the low stage, the less the rise required to bring the conditions into the opposite relation, and make fill take the place of scour.

This is verified at the wide shoal sections, by the well known fact that a minor rise coming after the channel has commenced to "cut out" will work positive disadvantage to the channel depth by causing "flattening."

On the other hand, supposing the river to have risen from the intermediate stage of equal area to bank full, and the section to have changed from the shaded to the broken outline, it will readily appear that the fill at the wide section will greatly exceed the scour at the narrow, and that the condition of equal area will be reached on the decline long before the stage falls to the level of the intermediate line.

Putting the results at the extremes together, it is proven that the vertical range of the crest of the shoal may be a very considerable fraction of the rise and fall of water surface. Also it is certain that the transition stage is variable, occurring earlier, both in rise and fall, as the preceding low or high stage has by joint influence of volume and duration adjusted the bed to the discharge of such volume with uniform velocity.

It has now been proven that

1st. There is a tendency to adjustment of the conditions of flow in the interest of uniform motion under which the stream continually reforms and rearranges its bed as the volume varies. If the volume remain constant for a sensible period, the adjustment to the conditions will be perfected and remain stable. But if the variations of volume be rapid, extreme, or irregularly alternating, the results are imperfect. Volume is the agent, but requires time to operate. Duration, therefore, measures the effect as much if not more potently than the actual range of volume.

2d. There must be a movement of material off from the shoals into the pools, and out of the pools upon the shoals as the stream rises and falls.

3d. This alternating movement between pool and shoal is the general

law governing the movements of silt along the bottom of rivers, irrespective in this connection of the mode in which the material is transported, whether it be by intermittent suspension, or by being swept along the bottom.

4th. Since this alternating movement follows the variation of stage, the higher the flood and the longer it continues, the more considerable the mass and the greater the height of deposit on the shoals, and the longer the low stage continues, the deeper and better the channels which will be cut through the shoals by a given volume.

The practical consequences of the movement, therefore, extend in two directions, which may or may not be harmonized. The one affecting the flow of the waters, or the discharge of floods, the other the navigability of the channels.

If the first of these be examined, the fact under discussion will be recognized as an efficient equalizer of velocity, adapted to the natural condition of great variation of volume in an unstable bed, by which the evil consequences of extremes are neutralized by their succeeding each other in a more or less perfect alternation, and the river guarded against extended radical changes by the forces being compelled to exhaust themselves upon local action. But, since the action at extremes of stage produces contrary results at the places where amelioration of conditions in the interest of navigability is most necessary, that is, the wide reaches, the increase of flood volume by concentration is opposed to the permanence of low-water channels.

Therefore the changes of wet section due to the variation of width harmonize with similar changes due to outline in demanding recognition of the principle, that variations of volume are to be lessened, if practicable, never designedly increased, by engineering works.

The alternating silt movement is the apparent cause of obstructed navigation in rivers whose beds are unstable; for, if the crests of the shoals continued throughout the year at their low stage elevation, the depth would, except for very short periods, be ample for all the requirements of commerce.

Considered in this limited view, the movement appears to be an evil, and the naturally suggested remedy is to obliterate the shoals by reducing the stream to an approximate uniformity of width. Assuming the practicability of doing this to any degree thought advisable, inquiry is pertinent concerning the physical consequences to the stream itself

when the proposed regulation is completed throughout the extent of a great river.

Attention has often been given to the modifying effect upon floods due to the vast volumes of water absorbed in channel filling. To reduce high-water widths to uniformity must materially lessen the reservoir capacity of the river-bed. To shut off lagoons and lateral swamps from the floods by embankment, must also intensify flood effects upon the lower parts of the river so narrowed and embanked. But this is not the point of the present inquiry, which turns rather upon the fact that the waters absorbed by channel and swamp filling during the rise, are given back, little by little, during decline, and so sustain and equalize the discharge. Impairment of the reservoir capacity of a river bed must therefore hasten and prolong the period of low water, and reduce the discharge below the natural limit, and so much the more as the application of such a system of improvement is extended. This suggests that while suppression of the alternating movement, through regulation of width at high stages, promises to remove the apparent cause of obstructed navigation, a worse evil may immediately appear as a direct consequence of such suppression, or rather of a suppression accomplished in this way. To avoid these undesirable secondary consequences of width regulation is an important practical problem, whose theoretical solution is readily found in equalization of volume as an attendant, logically, an antecedent step to regulation. This may not always be practicable ; but no engineer can question that the true principle of conserving and concentrating the natural forces of a river, in the interest of permanence, lies in the direction of equalization of volume.

The practicability of equalizing volume by retaining a portion of the flood waters in artificial reservoirs, to be drawn upon when the natural flow is insufficient for the needs of navigation or irrigation, has often been discussed, and in many cases set aside as unpromising. But if such a system was already constructed and in successful practical operation, without cost to any, a proposition to destroy or impair it should be received and entertained with great caution.

A reservoir system is naturally in operation upon all streams. To measure its importance requires the summation of the area of water surface presented by a great river and its permanent affluents, and an estimate of the volume required to fill the area to a depth equal to the

mean range between high and low water. This natural reservoir, if the bed is not subject to alternate changes, will be speedily drawn off and serve only to mitigate the floods and prolong the high stage. But if the bed is subject to alternation every shoal constitutes a dam, built up in time of flood, which slowly wasted during the decline renders each pool and bend a reservoir to sustain the low water volume, whose area is that of the water surface, and whose depth is the vertical range of the shoal or crest of bar, already shown to be, in rivers of unstable beds, a considerable fraction of the range of water surface.

This specially valuable reservoir action is the direct consequence of the variation of wet section under the special condition of width variable at high water.

Variation of the wet section has now been considered under two forms, one being the consequence of change of trace accompanying variation of volume, manifests itself in the transverse profile and affects stability of banks; the other being the joint result of variations of width and volume, manifests itself in the longitudinal profile and affects the navigable channel. The extent of both forms depends upon the mobility of the material composing the bed, and the duration of particular stages, up to the limit of complete adjustment or uniform motion. Alternation of scour and fill, lateral or longitudinal, is determined by variation of volume, and would be suppressed by equalization.

It has been shown that artificial uniformity of width tends to increase the variation of volume (strictly, it only brings the discharge into closer conformity to the varying supply). From the general argument it follows that stability of the condition of uniformity is endangered by such increased variation. Equalization of volume will bring about stable conditions of bed and banks, and, paradoxical as it may seem, approximate uniformity of width is among the stable conditions so brought into existence.

The general proposition that stable conditions of cross-section attend equalized volume is evidenced by the beds of lake-fed streams and artificial channels of regulated discharge. The fact of uniform width being among the stable conditions is evidenced only by the lower parts of the few silt-bearing rivers traversing an alluvial formation of homogeneous soil extending to depths practically unlimited, whose areas of drainage include such variety of climate as to afford an approximately equalized volume.

Seeking a theory of channel formation in accordance with the facts, it is submitted, that a stream freely forming its own channel in an alluvial plain will be governed by the following principles:

1st A plastic mass moving in a resisting medium will assume the form in which it encounters least resistance.

2d. The transverse section of a body adjusted to the form of least resistance will have the ratio of $\frac{\text{area}}{\text{perimeter}}$ a maximum.

3d. The form of a fluid mass will vary, whenever the direction of movement is changed, by virtue of the unequal moments of inertia in different parts of the mass.

4th. The form being unsymmetrical, and the direction of movement not a straight line, if the mass be variable, the path described by the center of volume or hydraulic axis will be variable also in position and length.

5th. With given limits of mass variation, the vagation of the paths will lie within a zone of certain width.

With these principles in mind, it is readily seen that a river traversing a homogeneous soil will form a bed whose width and depth will be largely determined by the variations of volume, being wider and shoaler as the vagation of hydraulic axis is greater; narrower and deeper, as the volume becomes constant and the vagation of hydraulic axis less. There is nothing in this conclusion which limits it to the lower parts of rivers. That constancy of volume is essential to stability of condition, may, therefore, be put among the principles of general application. This may not be practically realized, but is a valuable index of the direction engineering plans should take; for the converse is equally true. Changeable cross-section and excessive width are in nature inseparable from extreme and irregular variations of volume.

From the preceding partial study of the consequences of variation of wet section, a general conclusion is suggested which involves a more radical distinction than has heretofore been recognized between hydraulics as applied to large streams, and to pipes or other rigid channels.

Hydraulics, as generally understood, is founded upon experiments, and deductions therefrom, made upon channels with boundaries really unyielding, or supposed to be so, and treats velocity as a flexible quantity adapting itself to rigid conditions of area and volume. To some

extent in all rivers, but particularly in silt-bearers with unstable beds, uniform velocity is approximated, and areas become the flexible quantity, or, in a certain sense, the dépendant variable. With this important complication of the elements in view, the impossibility of applying the same formulas to the two classes of channels is obvious.

Moreover, since all the elements of area are independent variables, the impossibility of obtaining formulæ of practical value in terms of velocity, slope and hydraulic mean depths is suggested.

PLATE XXI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. IX Nº CCVI.
Mc MATHON VARIATION OF WET SECTION OF RIVERS

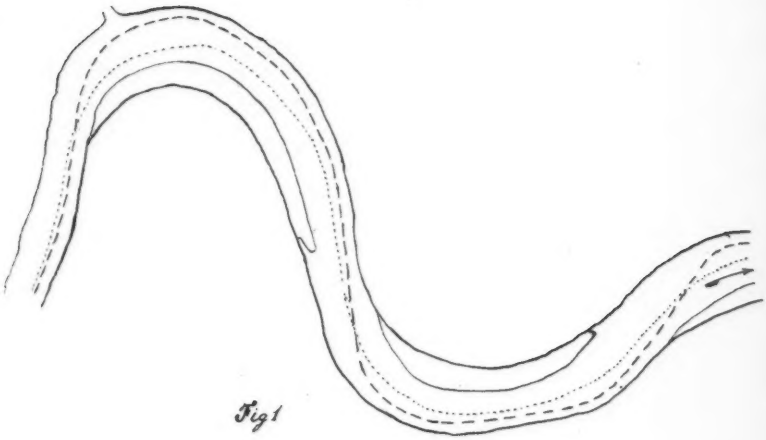

Fig 1

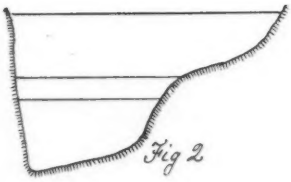

Fig 2.

Fig 3.

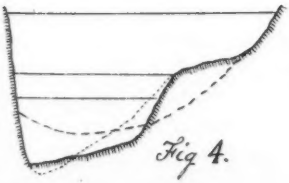

Fig 4.

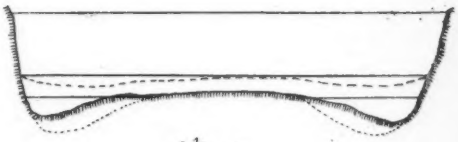

Fig 5.

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CCVII.

(Vol. IX.—October, 1880.)

WIND PRESSURE AGAINST BRIDGES.

By ASHBEL WELCH, Member of the Society.

READ AT THE TWELFTH ANNUAL CONVENTION, MAY 25TH, 1880.

With Discussion by C. SHALER SMITH, THEODORE COOPER, JAMES B. EADS and ROBERT BRIGGS.

The disaster to the Tay Bridge caused by the force of the wind, suggests a reconsideration of the rule by which that force on such structures is usually calculated.

In all the calculations I have been able to find, it is assumed that the pressure on a flat surface, at right angles to the wind, is equal to a column of air of the same base as that surface, and of the height that would give a falling body the actual velocity of the wind.

If the particles of air were annihilated instantly after impact, or if they were solids, and instantly dropped or rebounded out of the way, this rule would require but little qualification.

But the real conditions are entirely different from those just supposed. After impact, an additional amount of force pressing against the obstruction must be expended to move the intercepted air sideways out of the way. The air rebounds in divergent lines, and thus a base is pressed upon, broader than the surface of the solid, and yet supported by it. The solid surface may be conceived as a fulcrum supporting a leverage extending each way beyond itself. The pressure must be sufficient not only to stop the original motions of the air, but to generate the new motions that disturb the surrounding atmosphere.

Hence the pressure against the solid flat surface, especially if a narrow one, is always greater than that deduced from the rule in question.

A moving fluid presses upon a fixed obstruction, wide or narrow, continuous or interrupted, with just the amount of force which the obstruction stops. When it diverts any part of the fluid from its original course, I suppose the amount of force stopped is as the sine of the angle of divergence.

A stake set up in the sweep of driving sand, or snow, not only piles it up in front, but for a considerable distance on each side, showing that the stake stops or checks a column of air much wider than itself, and so stops the force and sustains the pressure of a column wider than itself.

It is the same in water. From Beaufoy's careful experiments it appears, that to move a short plank of 3 inches by 20 inches section, endways through the water with a velocity of 8 feet per second, required, besides the force necessary to overcome the friction, a force of 60.69 pounds. By the rule in question it should have required only 26.04 pounds. The actual resistance was two and a third times that which the rule gives. As air is elastic, we should expect that its impact would create greater disturbance around the obstruction than water, and so create a greater pressure in proportion to its velocity and weight.

An orifice through a thin side of a vessel filled with water, relieves that side of only about five-eighths of the pressure which would be upon the area of the orifice if closed up. So the difference between the pressure of the wind against a continuous plate, and against the bars of an iron bridge superstructure of the same height and length as the plate, is not measured by the extent of open space between those bars, but by the *vena contracta*, probably smaller in air than in water.

The area of surface of booms and bars of the superstructure of one

of the longest spans of the Tay Bridge is said to have been 800 square feet. All of the calculations of pressure I have seen are founded on the assumption I have mentioned, and so far as I have seen, that assumption has not been questioned. If the velocity of the wind was 150 feet per second (about 102 miles per hour) the pressure, on that assumption, was 28 pounds per square foot, which on 800 square feet makes 22 400 pounds. I believe with that velocity it was nearer 50 000 pounds.

This subject opens a fine field for investigation by some young scientific bridge building engineer. A very simple and easy experiment is to suspend a system of flat bars of iron or wood, so as accurately to face the wind, and by comparing the extent of its exposed surface, its weight, and deflection from the vertical, to deduce the force against it. Some thing of this kind must often have been done, but I do not think the results, whatever they may be, are generally known.

Discussion of the above Paper.

C. Shaler Smith.—I have taken pains to examine the plans of the Tay Bridge, and accurately calculate the wind surface exposed, by our American rules. Instead of 800 given by the English method, I find it to be 2 576 square feet. That is very easily tested by examination of the plans.

Theodore Cooper.—It is surprising, when we examine our knowledge of the pressure of the wind, to find how little we know, and how unreliable is that little. Engineers have almost universally relied upon the table of wind pressures and velocities first published by Smeaton, without stopping to examine into its reliability or how it was obtained.

On examining into past experiments upon the relation existing between the velocity of the wind and the pressure due to such velocity, we find they were nearly all made by means of rotating arms, with the resisting surfaces fastened at their extremities, the velocities being determined by the revolutions of the arms, and the resistances indirectly by determining the power exerted upon the mechanism. For the purpose of getting a better knowledge of the power of wind-mills, the more immediate purpose of many of these experiments, these experiments may have been well enough; but to rely upon them for the facts upon which to base the designing of our modern structures to resist the wind, seems very unscientific. For moderate velocities the results may have been approximately correct; but for the higher velocities, which are the

important ones for our present purposes, they would be very untrustworthy. The pressure exerted by the fans against air already more or less in motion by the rotation of the fans, would be very different from the pressure of a mass of air moving at the same high velocity striking against a stationary surface.

The anemometers now in use for meteorological observations only register the velocity of the wind, that being sufficient for the purpose of that science. We, as engineers, have taken these recorded velocities, and by means of the table before mentioned, determined the pressures we have thought it necessary to provide against. Moreover, we have considered the maximum velocities recorded at these meteorological stations as the maximum velocities against which it was necessary to provide.

Now, these stations are usually located at isolated and elevated points, and the observations aim to get the velocities of the general currents in motion over extended territories, rather than the level ones most important for engineering purposes. That a general wind current moving over the country at a velocity of say 20 miles per hour, would develop local currents of much greater velocity when the current is contracted by means of gorges, ravines or the high banks of rivers, seems very probable. At such points are frequently located our most important structures, and to what extent such a current may be increased in velocity by being thus confined, is very important for us to know, and it would be interesting to have comparative observations made upon such general currents and the same contracted by neighboring gorges.

During the existence of a prevailing wind it is not supposable that its velocity suddenly changes; but, due to its deflections, we experience an ever changing velocity, passing from the maximum to a dead calm, and frequently, by the rhythmic action of the deflections, we have recurrent gusts or impulses which repeat themselves almost regularly. These would produce upon many of our structures an effect much greater than that due to the same current applied steadily, did it happen that the period of recurrence of these gusts corresponded to the period of oscillation of the structures, for the effect would then be accumulative.

When considering the deflection of a wind current, the up-lifting or down-pressing action of the wind is readily understood. The wind seldom moves in horizontal lines, and it is as important frequently to consider the oblique, as the horizontal action of the wind.

In addition to the want of a proper knowledge of the relation between wind pressure and velocity, and of the maximum velocities of local currents, our knowledge is very limited in regard to the relative effects on different sized or shaped surfaces, and to what extent one surface can form a lee for another placed behind it. Will the resistance offered by a plane surface be the same as a surface of the same area with ribs or projections upon it? How will the resistance of a grated or latticed surface compare with a plane surface? How close must one plate be to another in front of it to be exempt from any additional resistance? And how far off, compared to their size, must they be to get equal resistances? The importance of this knowledge is evident when we see that it may, possibly, quadruple the surface usually considered as affected by the wind pressure. It has been suggested that General Myer, Chief of the Signal Service, would have facilities for making many or all of the desired investigations, and a proper representation by this Society of the needs of engineers in regard to this subject, would have great influence in bringing him to consider the matter. Would it not be well to discuss among ourselves the proper class of experiments desired, and how they could be made with the least expense, and then lay the matter before him for his consideration?

James B. Eads.—I think, upon this subject, there is one point that has not been touched upon, which is very important. It has been generally assumed that wind pressure should be taken as 30 to 50 pounds to the square foot in this country, without reference to the height of the bridges. Now the velocity must be very much greater as you get away from the earth, and consequently a pressure which may be safe at 50 pounds to the square foot within 100 feet, or even 50 feet above the surface of the ground, is not safe when you get 100 or 200 feet above the surface. That is a matter engineers should take into consideration in providing for wind pressure. I do not remember ever having seen attention called to that point. It strikes me as a very important point. Of course that is with reference to resistance to gales and hurricanes, as tornadoes sometimes exert such enormous force close to the earth. It may be stated that their action is limited generally to a very small space, and where it is not possible, or where it is not proper to incur expense in providing against such tornadoes as lifted a locomotive off the track near the eastern end of the St. Louis bridge, you may safely assume that the

chances of tornadoes striking a bridge are infinitely small. The tornado there absolutely lifted the locomotive off the rails, on the leeward side of the locomotive, which of course would have been started by the flanges of the wheels if it had not been lifted; they were undisturbed and the locomotive was thrown over into a ditch or depression some six feet lower than the rails. Col. Flad, and some other gentlemen engaged with me at the time when the St. Louis bridge was being built, made an examination of it.

C. Shaler Smith.—I may state, in reference to tornadoes and their limited area, that I recently went over the track of the Marshfield tornado in this State with Prof. Nipher. We struck into the track of the tornado and he followed it back to where it commenced, while I followed it forty-six miles till it left the ground. The track of the tornado was about 1 800 feet wide clear, leaving the ground at no point in this forty-six miles. It developed at one place a lifting power of 58 pounds per square foot. I found no place where I could get an accurate measurement of its horizontal force. You can judge of its vertical force from the fact that it lifted a piano and moved it 270 feet, and placed it on its feet again without any serious detriment to the piano, while of the house from which it was taken, built with a 13-inch wall, a five room house—one of these large country places—no brick was left standing two feet from the ground. That I think is the widest tornado belt I know of. That at St. Charles was, I think, not more than about sixty feet wide; it developed at one point a lifting power of 85 pounds to the square foot. Its course was very irregular. The central point of this column swung to and fro over a space of 2 000 feet. As it moved from the southwest to the northeast, that tornado passed through the center span of the St. Charles bridge, taking up a barrel of tar that stood in the middle of the bridge, bursting the barrel all to pieces and smearing the tar all round the posts. The only thing that saved the bridge was the narrowness of the column. If it had covered 200 feet instead of sixty it would have carried away the west span. The same storm destroyed the St. Charles jail, damaged the court house and a large brick house. Afterwards I noticed that wherever the tornado passed over a house, the walls of the house as a general thing fell out, with the exception possibly of the gable end,

showing that there was a vacuum and it was the expansion of the internal air that produced the destruction.

Robert Briggs.—However unprepared I may be to speak upon the action of wind upon the Tay Bridge, this question of the tornado and its causes is something of which I can speak. The conditions of the atmosphere that establish a tornado are exceedingly simple. First, There is required nearly an absolute calm—a cessation of wind from all directions; that it shall be a clear day in summer, when the rays of the sun shall be effective in imparting heat to the surface of the ground which shall be communicated to the stratum of air immediately in contact with the ground. Under these conditions the lower stratum of air becomes in very unstable equilibrium, as it will underlie the great mass of the atmosphere through which the sun's rays will have passed without heating or expanding the same in the passage. The lower stratum of light air is therefore compressed by a layer of more dense air uniformly distributed upon its surface, and its effort to rise is resisted by this layer, and by the adhesion of the air to the surface of the earth. In the conditions of quiet supposed, the tendency of the heated air to rise is purely a vertical one. Having supposed or rather having established the foregoing status of equilibrium over a considerable extent of territory, a vertical movement must (in the absence of some sufficient cause for a general horizontal current over the entire territory) eventually disturb it, and at some point more or less central to the heated stratum, an ascending current will generate, slowly at first, but increasing in velocity as the compressed stratum supplies air from all directions with accelerating force towards the vacuity occasioned by continued upward flow. Once commenced, the flow of ascending current will be reinforced and maintained in the one locality (unless other causes intervene) until the entire heated stratum is displaced by normal air, during which displacement or substitution there will exist upon the surface of the ground a rush of air from every quarter, directed to a common center. If the rates of flow from all quarters were altogether uniform, and if the directions of flow were to a common center, the result at the center of action would merely make an inverted funnel of air with a fixed axis or position—a column of air moving directly upwards at high velocity, which column would have the contour due to the least resistance of the particles of air in changing their direction from that of horizontal flow parallel to a plane

surface, to that of vertical flow parallel or nearly so to a vertical axis. The dimensions of such a column are dependent upon the velocity and volume of the air, and the contour would be that of the *vena contracta*.

The flow of horizontal currents towards a common center is, however, one of very unstable equilibrium. Some disturbing influence always directs this flow upon one or the other side of the central axis, and the currents move inward on a tangential path, establishing a whirlwind. The interior of the inverted-conoid-form of the column in contact with the earth, in place of being composed of air under the compression due to the velocity of the winds from all sides, actually has a vacuity caused by the centrifugal action of the whirling currents, and the external dimension of this conoid (or its diameter on any given plane) is much increased by this action.

This proposition can be comprehended by observation of the efflux of water from a common wash-basin with drain plug, or from any vessel for the discharge of water with orifice at the bottom. The formation of the whirlpool, the depression of the level of the water surface due to centrifugal action and to a change of direction of the currents from the horizontal to the downward direction, can be at once seen. In fact in the case of discharge under some conditions of form, we can get a hollow tube of water instead of a solid stream, the vacuity arising from the efflux being carried downward.

The actual pressure of air, or more properly vacuum, from a tornado is not great. The pressure stated by the gentleman preceding me did not exceed 58 pounds to the square foot, only $\frac{4}{10}$ths of a pound per square inch. Comparing this with the pressure of the atmosphere or 14.7 pounds per square inch, the vacuity appears very small. The second tornado pressure quoted was only 85 pounds per square foot, or $\frac{6}{10}$ths of a pound per square inch. Even this is a very small part of an absolute vacuum. These pressures of 58 and of 85 pounds are, I suppose, deduced values from some movement of some object, but it is proper to state that they are probably very much under the actual pressures existing if not exerted. It is only needful to consider that the atmospheric pressure in ordinary conditions is 2 117 pounds per square foot to appreciate that there exists what the newspaper reporters call an "irresistible force" in the disturbance of that pressure.

The locality of the axis of any vortex or tornado is not necessarily, in fact is never, a fixed one, but it will be found to have a path or direction

often tortuous from local influences or obstructions, but generally following some sweep of curvature of great radius, rather than a straight or sinuous line. It is very possible that in places favorable to the repetition of tornado storms, their paths will be found to have some usual direction that will allow a probable course to be asserted as the path after commencement. The phenomena of a tornado or a whirlwind are thus seen to be by no means incompatible with the well understood physical properties of the atmosphere.

The most remarkable tornado, the results of which I have had the opportunity of observing personally, was one that occurred in 1850 or 1851, at Medford, near Boston. This tornado had a course in its entire length of about 13 miles. The path of its center or nucleus of absolute devastation was only about 18 feet wide. The general path of this tornado was a comparatively straight line for two or three miles, although the track itself was extremely tortuous. It was in a country of stone walls and of boulder rocks. There were houses and trees in the course of its track. The stones, the walls, the trees, the houses were completely swept away or displaced.

It is in similar limited tracts that any tornado exerts its force.

I wish it to be thoroughly appreciated that there is no mystery about the formation of a tornado; it is merely the result of an unstable equilibrium that exists from the levity of air on the surface of the ground when there is general quiescence of the atmosphere.

Our hail storms and sudden summer showers have similar origin. The levity of our summer air arises not alone from heat but in part from the moisture present. With the presence of water surfaces and of vegetation, the proportion of moisture at our high temperature for summer becomes large. In the case of the hail storm or summer shower, it is the condensation of this moisture which supplies the hail or the rain precipitated. The same conditions of levity of air upon the surface as have been described for the tornado, will have established a less intense and possibly more voluminous local upward current of air. At the height of three miles—only three miles away—this current must have been rarefied and expanded to the point of not only condensation but of frigeration of all the moisture present. It is only necessary that there should be a gust of air from the ground upwards to this height, to make all the conditions for a hail storm on any summer day.

One of the providential things in the falling of hail stones, is that

they generally descend against a violent upward current of air, and are thereby deprived of the high velocity which such masses would attain in still air or in air in horizontal movement.

It must not even be admitted that these propositions as to the results of vertical currents of air relate alone to tornadoes or hail storms. In all cases and at all times, a rain storm may be taken as an indication of an ascending current of air; slowly ascending perhaps, but still ascending until some point of elevation is reached where the moisture present is incompatible with the temperature and pressure of the air. At five miles height the air is nearly anhydrous.

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CCVIII.

(Vol. IX.—October, 1880.)

CHEAP TRANSPORTATION VS. RAPID TRANSIT AND DELIVERY.

By MARTIN CORYELL, Member of the Society.

READ AT THE TWELFTH ANNUAL CONVENTION, MAY 25TH, 1880.

To open the subject of the proposed paper, it will be assumed that the productions of our country, and the manufactures, began to assume commercial importance about the year A. D. 1800.

The ocean, bays and rivers then formed the cheap transportation lines for commerce and travel; and, as a consequence, the largest cities, towns and trade centers were on navigable waters.

As the country increased in population, and agriculture advanced, and manufactures prospered, yielding a surplus above the wants of the producer, centers of trade and exchange were established in the interior of the country, and such towns as Worcester in Massachusetts, Albany and Schenectady in New York, Lancaster and Reading in Pennsylvania, Frederick in Maryland, Charlotte and Lynchburg in Virginia, were established, and were connected by roads with the cities on the navigable waters.

Then came the necessity for turnpikes, to cheapen, or rather to hasten, transportation; they were such a success over the common roads that the leading "Turnpikes" became over-burdened, and were nearly destroyed by the traffic they had to carry, and in connection with oppo-

sition and competing lines the tolls could scarcely keep them in repairs, and no dividends accrued for the original stockholders.

The great "National Road" or turnpike of the United States Government is familiar to older members.

The capital invested in turnpikes failed to be profitable. Yet the cities and the country adjacent to them profited greatly by their construction.

The next effort to meet the wants of a growing commerce, and to convey large amounts cheaply, was by the means of canals; and the completion of the Erie Canal established an era in canalling which pervaded the States of our Union, and many States plunged recklessly and —for that day—fearfully into debt to construct cheap transportation lines.

Of the many canals projected, and some completed, only a few remain that are useful to the country, and they are scarcely profitable after renewals, repairs and expenses are paid.

For about fifteen years canals seemed to be a favorite mode of transportation, and shares of stock were held above par; and water-fronts, accessible to canal boats and shipping, commanded fabulous rents; and merchandise, coal and lumber were piled from twenty to forty feet in height on the wharves.

With the canals, and the increase of trade and commerce they effected, were developed the difficulties and expenses of terminal facilities; and so great and so inconvenient were they that railroads became a relief, and actually grew into favor, to the detriment of canals and water-fronts.

The Philadelphia, Reading and Pottsville Railroad, the Hudson River Railroad, the Philadelphia Wilmington, and Baltimore, and the Baltimore and Ohio Railroads became the early antagonists of canal, river and bay navigation.

Many of us can remember how learnedly the late Charles Ellett proved that within a very few years the Reading Railroad would have to give way to the Schuylkill Canal. It would be abandoned, and the canal would be carrying the freight so cheaply that the railroad could not be maintained.

When some bold capitalists, with our venerable and esteemed Honorary member John B. Jervis, as their engineer, projected and estimated the cost of a railroad along the Hudson river, and the estimated amount of traffic and travel there would be to sustain it, many can re-

call the contempt and scorn with which canal engineers and vessel owners treated the subject, and thought it preposterous to entertain for a moment the possibility of a railroad contending with the free navigation of such a noble and magnificent river as the Hudson, with a capacity to accommodate the tonnage of the United States, and with floating palaces for the transmission of passengers to the number of a thousand or more at a time, at a speed of sixteen miles per hour.

The Camden and Amboy route, between New York and Philadelphia, consisting of thirty miles of steamboat travel to Amboy, twenty-eight miles of railroad to Bordentown, then twenty-six miles by steamboat down the Delaware to Philadelphia, has so long been abandoned for a through rail route, that it is almost forgotten.

So with the water route from Philadelphia to Baltimore, viz.: By steamboat down the Delaware river, forty miles, to Newcastle; then by rail fourteen miles, across the State of Delaware to Frenchtown; then by steamboat, on the Chesapeake Bay, about fifty miles, to Baltimore.

The Philadelphia, Wilmington and Baltimore through rail route used up this cheap and pleasant mode of transportation and travel.

These are a few leading instances in which water routes, giving cheap transportation to travel and commerce, had to yield to railroad lines.

No reference need be made to the rivers of the West and the great lakes, for the subject is familiar to all who have to do with commerce, travel and transportation.

The cities on tidal waters, Western rivers and lakes, have to avail themselves of railroad facilities of the most costly character, to maintain their commercial importance, and capital is most lavishly expended to secure this object.

Other competing elements to cheap transportation by water routes are the telegraph and telephone lines.

Cheap transportation by water in this country has not been, and cannot be, more than a temporary success. What has been done, and what now remains, has accomplished the mission intended, viz., an advancing step in the improvements and growth of our country and commerce.

All such routes have been virtually retired from the scene of competition and rivalry of the railroads.

The Suez Canal is put forward as a grand accomplishment in facilitating commerce, and so it is, and is deserving of all the honor and praise bestowed upon it and its projectors and engineers that executed the

work; and yet, within forty years or less, its commercial value will be rendered almost worthless by competing rail routes from India to England and France, by way of the Dardanelles and roads on either side of the Red Sea to the Mediterranean.

The wants of the day are rapid transit and quick delivery, and if that can be accomplished, cheap transportation must yield, and commerce can well afford, and will always cheerfully pay the difference of cost.

The commercial enterprise and knowledge of the laws and requirements of trade, which have of late years been incorporated in the management of large railroad enterprises, have made wonderful changes and necessities, that are far in advance of the conceptions and ideas of the law-makers and engineers of forty years ago.

Then, railroad charters gave a length of from 30 to 300 miles for the road, with the limit of about an acre of ground in the cities for terminal purposes, and the capital was about $30 000 per mile of road.

At the present time the extent of the leading railroad lines is so great, and the amount of capital invested is so enormous, that *Poor's Manual* of railroads has to be referred to for information.

Terminal facilities, rapid handling of freights, and no delays in transit, are now engrossing the attention of railroad officials, and the demand for expedition in the handling of grain and coal by the aid of steam machinery, through the grain elevators, and over coal trestles, the transportation and storage of oil, etc., is made evident from the costly structures that are found at all points, connecting rail transportation with shipping, or receiving and storing depots.

All goods in large amounts must be handled promptly and in good order, and machinery and facilities must be provided for so doing.

Some years ago the idea prevailed that large vessels would cheapen transportation.

The *Great Eastern* steamship was constructed in England, and Donald McKay, of Boston, built the large sailing vessel, *The Great Republic*, and there was a very large vessel constructed, to fill the area of the double or enlarged locks of the Delaware and Raritan Canal, to ply between New York and Philadelphia.

So much time and labor was required to discharge cargoes, to reload, distribute and secure freight, that the wharfage, demurrage and daily expenses whilst laying at their berths, completely absorbed all profits;

and that they were not a success may be inferred from the fact that no other vessels of this large class have been constructed since.

Commercial countries that are separated from each other by the oceans do not require immense ships, but of a size and speed that can make quick trips, and can enter ordinary rivers and harbors.

One large city, or one country, cannot monopolize the ocean commerce of the world.

The lading the largest vessels can carry must be distributed to manufacturers, or the people who consume or use, and they are not all on the shore of one great city that the ships can reach; therefore, the more numerous the commercial ports, the greater the opportunity to distribute the lading of the ships to the points required.

By examining the net work of railroads of this country and their power of rapid distribution of merchandise, we must be convinced that railroads do most essentially fulfill that duty, and make every section of our country important to commerce and trade.

Continental lines of railroads and telegraphs across the continent, from ocean to ocean, completed, under construction and projected, will take up and distribute all the lading the shipping may find to convey across either ocean.

If through freight, it will be carried so quickly that the extra cost of handling will be fully compensated for in time saved.

The members of this Society are fully cognizant of the several proposed canal routes across or through the continent, at or near Darien.

The instructive maps that have been exhibited, and the able papers which have been read, discussed, printed and circulated, prove the interest manifested in the proposed work, and the more the subject is investigated, the estimated cost of construction increases, and the estimated revenue decreases.

The experience had in this country is against canals.

It is not to be supposed that commerce can be diverted so far out of direct routes, merely to enjoy a great deal of cheap transportation and the pleasure of passing slowly through a lake, with locks for ingress and egress and very bad harbors, or a canal without locks, with a thorough cut through the Cordilleras, and a voluminous river, with often a hydrostatic pressure of sixty feet altitude threatening to work devastation to the canal and all that may be within it, and with a harbor on the Atlantic side much exposed.

Perhaps, through a tidal canal, with safe harbors on each ocean and a tunnel of seven miles in length; or by another projected scheme, with an artificial lake made by constructing a dam eighty feet in height, and reaching the proposed lake by four locks of twenty feet lift each, and leaving it by the same number of locks, and steam to do the work of opening and closing the lock gates, and taking vessels in and out of the locks.

An enterprising Quaker citizen of Philadelphia, Josiah White, publicly made the expression, "that rivers were made to be dammed," and through the influence of his moneyed friends, he was furnished with the means of placing dams in the Lehigh river, from twenty to over fifty feet in height.

For a time they were a success, but the guard banks settled and were worn down, the exposed wood work became decayed, and an extraordinary flood, "peculiar to American rivers" (as stated by Mr. Welch), came, and all but three of these pools created by the high dams and embankments were washed out and emptied, causing sad destruction of property and loss of life.

The high dams were not rebuilt, neither was the slack water navigation renewed, but a railroad was substituted.

Captain Eads, whose success in establishing and maintaining a ship channel at the mouth of the Mississippi has established for himself a reputation that is widely respected, and he proposes to supplant the Isthmian Canal by a scheme to convey ships from ocean to ocean by railroad.

Canals, and improvements of water-courses to a limited extent for commercial purposes, when judicially planned, are appropriate and proper.

When schemes, as a canal to unite the Ohio river with the Atlantic ocean, the lakes with the Mississippi river, a canal around Niagara Falls, or a canal to connect the Atlantic with the Pacific ocean, are advocated, it is an evidence that the advantages of railroads over canals, for profitable and successful transportation, are not fully comprehended.

The wonderful progress and success in bridge building, and the power and speed of the locomotive, have placed railroads and rail transportation far in advance of canals, and the canals, wherever located, simply supply a connecting link between the past and the present.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CCIX.

(Vol. IX.—October, 1880.)

THE CRIPPLING STRENGTH OF WROUGHT IRON COLUMNS.

By C. L. GATES, JUNIOR of the Society.

READ AT THE TWELFTH ANNUAL CONVENTION, MAY 25TH, 1880.

From experiments made by G. Bouscaren, C. E., and from former tests of his own upon wrought iron columns, C. Shaler Smith, C. E., devised formulæ applicable to the several forms in use, approximating quite closely to the results of their experiments in most instances. These formulæ were adopted by the Engineer Department of the Chicago, Milwaukee and St. Paul Railway several years ago, and since their publication have been quite extensively used in specifications of works throughout the country and somewhat abroad.

The purpose of this paper is to present them to the Society in such shape as to invite criticism as to their correctness.

Said formulæ herewith given, show the ultimate crippling strength in pounds per square inch of section of the several forms of posts, H. representing the length between end bearings in terms of the least diameter.

From these formulæ I prepared tables for ultimate crippling strength for the various columns from 15 to 60 diameters long, which are here given in tables I., II. and III.

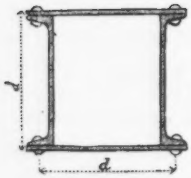
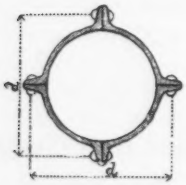
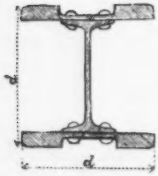
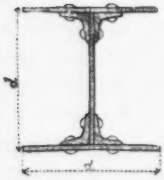
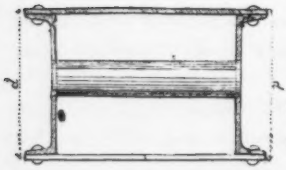
	Flat Ends.	One Pin End.	Two Pin Ends.
SQUARE COLUMN.			
	$\frac{38{,}500}{1+\frac{H^2}{5820}}$	$\frac{38{,}500}{1+\frac{H^2}{3000}}$	$\frac{37{,}500}{1+\frac{H^2}{1900}}$
PHŒNIX COLUMN.			
	$\frac{42{,}500}{1+\frac{H^2}{4500}}$	$\frac{40{,}000}{1+\frac{H^2}{2250}}$	$\frac{36{,}600}{1+\frac{H^2}{1500}}$
AMERICAN COLUMN.			
	$\frac{36{,}500}{1+\frac{H^2}{3750}}$	$\frac{36{,}500}{1+\frac{H^2}{2250}}$	$\frac{36{,}500}{1+\frac{H^2}{1750}}$
COMMON COLUMN.			
	$\frac{36{,}500}{1+\frac{H^2}{2700}}$	$\frac{36{,}500}{1+\frac{H^2}{1500}}$	$\frac{36{,}500}{1+\frac{H^2}{1200}}$
COMMON CHORD SECTION.			
	The pin being so placed that the moment of inertia is, as near as practicable, equal on both sides of same, use formulæ for square column.		

TABLE I.

Ultimate Crippling Strength per Square Inch of Columns with Flat End Bearings.

H = Length of column in terms of least diameter.	Square column.	Phœnix column.	American column.	Common column.
15	37 067	40 476	34 434	33 693
16	36 876	40 212	34 167	33 339
17	36 679	39 935	33 888	32 971
18	36 470	39 645	33 597	32 589
19	36 252	39 343	33 294	32 195
20	36 024	39 030	32 982	31 790
21	35 785	38 706	32 659	31 375
22	35 544	38 373	32 327	30 952
23	35 292	38 029	31 987	30 520
24	35 033	37 677	31 640	30 082
25	34 767	37 317	31 285	29 639
26	34 492	36 949	30 925	29 191
27	34 212	36 575	30 559	28 740
28	33 929	36 192	30 188	28 286
29	33 639	35 807	29 813	27 831
30	33 344	35 424	29 435	27 375
31	33 044	35 021	29 054	26 919
32	32 740	34 621	28 671	26 463
33	32 431	34 219	28 285	26 010
34	32 120	33 813	27 899	25 557
35	31 806	33 406	27 512	25 108
40	30 198	31 352	25 584	22 919
45	28 562	29 310	23 701	20 857
50	26 932	27 321	21 900	18 952
55	25 333	25 415	20 208	17 214
60	23 787	23 611	18 621	15 643

TABLE II.

Ultimate Crippling Strength of Columns with one Pin End.

H=Length of column in terms of least diameter.	Square column.	Phœnix column.	American column.	Common column.
15	35 814	36 364	33 182	31 739
18	34 748	34 957	31 898	30 016
20	33 971	33 962	30 991	28 815
21	33 566	33 445	30 518	28 207
22	33 152	32 919	30 038	27 596
23	32 729	32 386	29 552	26 984
24	32 299	31 847	29 061	26 373
25	31 862	31 304	28 565	25 765
26	31 420	30 759	28 067	25 161
27	30 973	30 212	27 568	24 563
28	30 523	29 664	27 068	23 971
29	30 070	29 117	26 569	23 387
30	29 615	28 571	26 071	22 812
31	29 153	28 029	25 576	22 247
32	28 703	27 489	25 084	21 692
33	28 246	26 948	24 590	21 147
34	27 794	26 424	24 112	20 614
35	27 337	25 899	23 638	20 092
36	26 885	25 381	23 160	19 582
37	26 424	24 869	22 693	19 083
38	25 990	24 364	22 232	18 597
39	25 547	23 866	21 778	18 123
40	25 109	23 377	21 331	17 661
45	22 985	21 053	19 210	15 532
50	21 005	18 947	17 290	13 688
55	19 170	17 062	15 569	12 100
60	17 500	15 349	14 038	10 737

TABLE III.

ULTIMATE CRIPPLING STRENGTH OF COLUMNS WITH TWO PIN ENDS.

H = LENGTH OF COLUMN IN TERMS OF LEAST DIAMETER.	SQUARE COLUMN.	PHŒNIX COLUMN.	AMERICAN COLUMN.	COMMON COLUMN.
15	33 798	31 825	32 342	30 666
20	31 226	28 894	29 709	27 375
21	30 679	28 283	29 153	26 691
22	30 126	27 670	28 592	26 010
23	29 568	27 057	28 028	25 333
24	29 007	26 444	27 462	24 662
25	28 444	25 834	26 895	24 000
26	27 880	25 229	26 329	23 348
27	27 318	24 629	25 766	22 706
28	26 759	24 036	25 210	22 077
29	26 202	23 451	24 653	21 460
30	25 650	22 874	24 104	20 857
31	25 103	22 307	23 561	20 268
32	24 561	21 750	23 026	19 694
33	24 028	21 204	22 499	19 135
34	23 501	20 669	21 980	18 591
35	22 982	20 146	21 471	18 062
36	22 472	19 635	20 970	17 548
37	21 970	19 135	20 479	17 050
38	21 477	18 647	19 999	16 566
39	20 994	18 172	19 528	16 097
40	20 520	17 709	19 067	15 643
45	18 298	15 574	16 920	13 581
50	16 326	13 724	15 029	11 838
55	14 583	12 132	13 377	10 367
60	13 058	10 766	11 939	9 125

Plate XXII. is a graphic representation of the strength of these columns in which the abscissa represents length of columns and the ordinates strength.

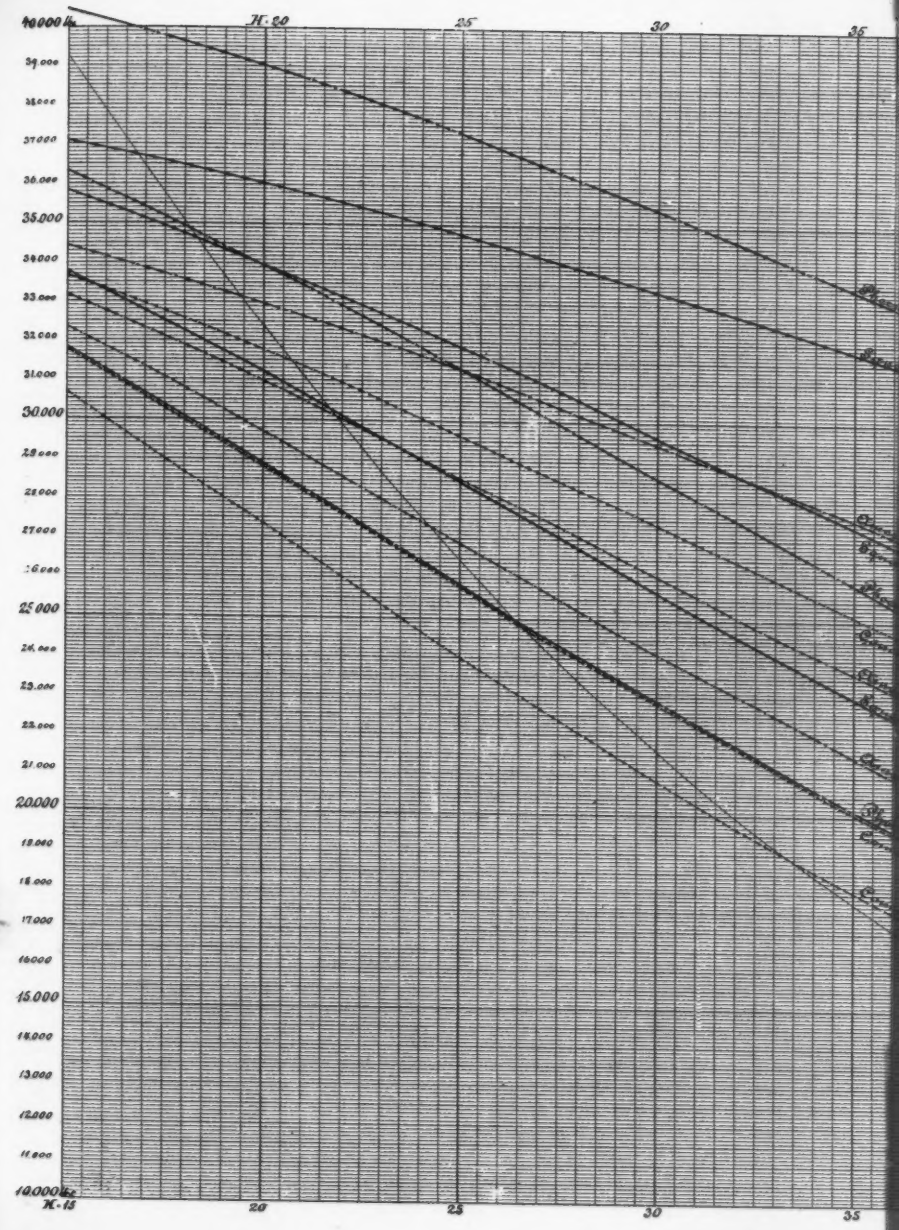
From the fact that in all built columns the liability of imperfections increases to some extent with the length of the column, a sliding factor of safety is introduced.

The following tables have been calculated, using a factor of safety of $4 + 0.05\,H$.

TABLE IV.

SAFE CRIPPLING STRAINS. COLUMNS WITH FLAT ENDS.

H=LENGTH OF COLUMNS.	SQUARE COLUMN.	PHŒNIX COLUMN.	AMERICAN COLUMN.	COMMON COLUMN.
15	7 822	8 521	7 249	7 093
16	7 683	8 377	7 118	6 946
17	7 563	8 234	6 987	6 798
18	7 443	8 091	6 856	6 651
19	7 324	7 948	6 726	6 504
20	7 205	7 806	6 596	6 358
21	7 087	7 665	6 467	6 213
22	6 970	7 524	6 338	6 069
23	6 853	7 384	6 211	5 926
24	6 737	7 245	6 084	5 785
25	6 622	7 110	5 959	5 646
26	6 508	6 971	5 835	5 508
27	6 396	6 836	5 712	5 372
28	6 284	6 702	5 590	5 238
29	6 173	6 570	5 471	5 107
30	6 063	6 440	5 352	4 977
31	5 954	6 310	5 235	4 850
32	5 846	6 182	5 119	4 726
33	5 740	6 056	5 006	4 604
34	5 635	5 933	4 894	4 483
35	5 531	5 810	4 789	4 367
40	5 033	5 226	4 264	3 820
45	4 570	4 690	3 792	3 337
50	4 143	4 203	3 369	2 916
55	3 728	3 765	3 004	2 550
60	3 398	3 373	2 660	2 235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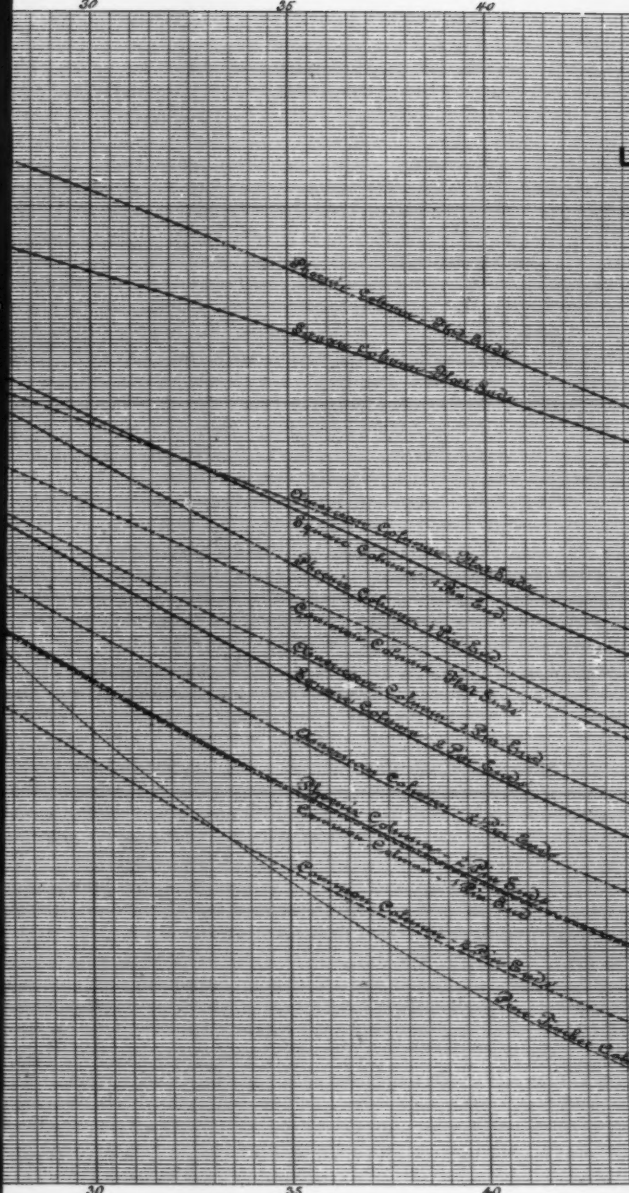
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40
30
35
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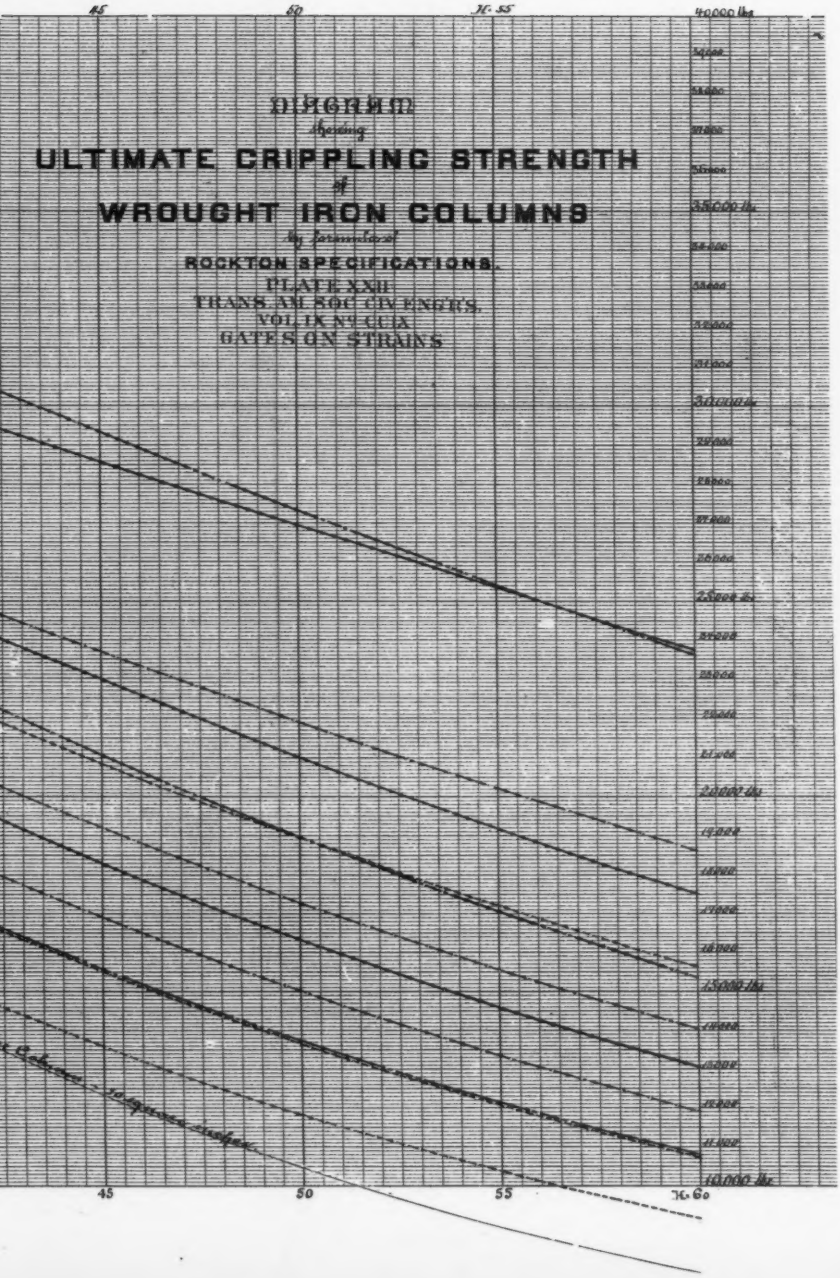
DIAGRAM
showing
ULTIMATE CRIPPLING STRENGTH
of
WROUGHT IRON COLUMNS
by formulas of
ROCKTON SPECIFICATIONS.
PLATE XXII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. IX. No. CCIX
GATES ON STRAINS
45
50
55
K. 60
40000 lbs
35000 lbs
30000 lbs
25000 lbs
20000 lbs
15000 lbs
10000 lbs

TABLE V.

Safe Crippling Strains. Columns with One Pin End.

H=Length of Column.	Square Column.	Phœnix Column.	American Column.	Common Column.
15	7 540	7 656	6 986	6 682
18	7 091	7 134	6 510	6 126
20	6 794	6 793	6 193	5 763
21	6 647	6 623	6 043	5 586
22	6 500	6 455	5 890	5 411
23	6 355	6 289	5 738	5 240
24	6 211	6 125	5 589	5 072
25	6 069	5 963	5 441	4 908
26	5 928	5 804	5 296	4 747
27	5 789	5 647	5 153	4 591
28	5 652	5 493	5 013	4 439
29	5 588	5 343	4 875	4 291
30	5 385	5 195	4 740	4 148
31	5 253	5 050	4 608	4 008
32	5 125	4 909	4 479	3 874
33	4 999	4 770	4 352	3 742
34	4 875	4 636	4 230	3 616
35	4 754	4 504	4 110	3 494
36	4 635	4 377	3 993	3 376
37	4 519	4 251	3 879	3 262
38	4 405	4 129	3 768	3 152
39	4 294	4 011	3 660	3 046
40	4 185	3 896	3 555	2 944
45	3 678	3 368	3 074	2 485
50	3 232	2 915	2 660	2 106
55	2 840	2 528	2 307	1 793
60	2 500	2 198	2 006	1 534

TABLE VI.

Safe Crippling Strains. Columns with Two Pin Ends.

H = LENGTH OF COLUMN IN TERMS OF LEAST DIAMETER.	SQUARE COLUMN.	PHŒNIX COLUMN.	AMERICAN COLUMN.	COMMON COLUMN.
15	7 115	6 700	6 809	6 471
20	6 245	5 779	5 942	5 475
21	6 075	5 601	5 773	5 285
22	5 907	5 425	5 608	5 100
23	5 741	5 254	5 442	4 919
24	5 578	5 085	5 281	4 743
25	5 418	4 921	5 123	4 572
26	5 261	4 760	4 968	4 405
27	5 106	4 604	4 816	4 245
28	4 955	4 451	4 668	4 088
29	4 807	4 303	4 524	3 938
30	4 664	4 159	4 383	3 792
31	4 523	4 019	4 245	3 652
32	4 386	3 884	4 112	3 517
33	4 253	3 753	3 982	3 387
34	4 123	3 626	3 856	3 262
35	3 997	3 504	3 734	3 141
36	3 875	3 386	3 616	3 026
37	3 756	3 271	3 501	2 915
38	3 640	3 161	3 390	2 808
39	3 529	3 054	3 282	2 705
40	3 420	2 952	3 178	2 607
45	2 928	2 492	2 707	2 173
50	2 512	2 111	2 312	1 821
55	2 160	1 797	1 982	1 536
60	1 866	1 538	1 706	1 304

Diagrams, as before constructed, are added, representing graphically actual compressive strains permitted on wrought iron columns as per given formulæ. (See Plate XXIII.)

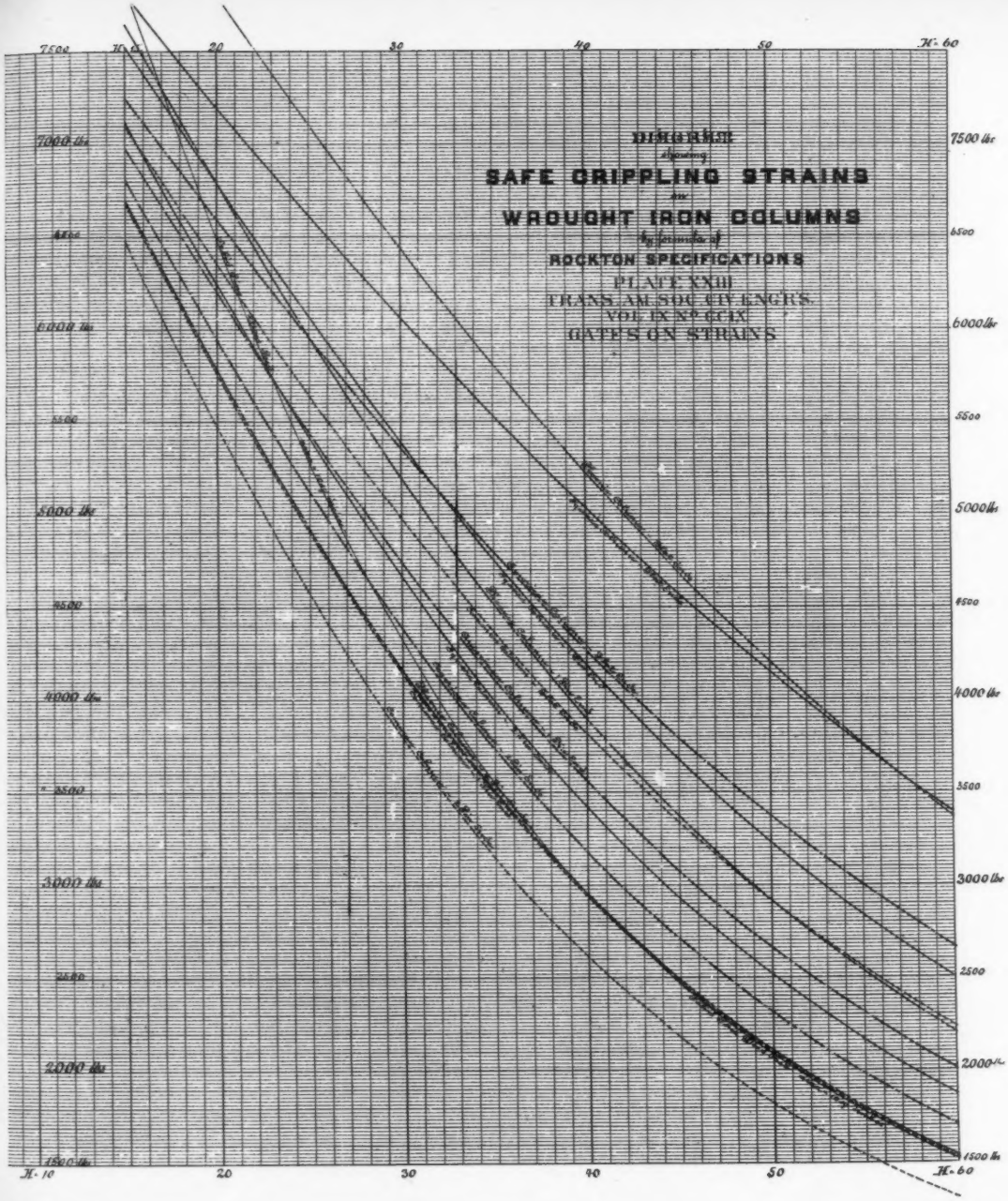
DIAGRAM
showing
SAFE CRIPPLING STRAINS
in
WROUGHT IRON COLUMNS
by formula of
ROCKTON SPECIFICATIONS
PLATE XXIII
TRANS. AM. SOC. CIV. ENGRS.
VOL. IX. No. CCIX.
GATES ON STRAINS
7500 lbs.
7000 lbs.
6500
6000 lbs.
5500
5000 lbs.
4500
4000 lbs.
3500
3000 lbs.
2500
2000 lbs.
1500 lbs.
H-10
20
30
40
50
H-60